VOLUME 84 NO. SA4
JULY 1958
PART 1

JOURNAL of the

Sanitary Engineering Division

PROCEEDINGS OF THE

AMERICAN SOCIETY
OF CIVIL ENGINEERS

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Journal of the

SANITARY ENGINEERING DIVISION

Proceedings of the American Society of Civil Engineers

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Paper 1701 July, 1958 SA 4

Journal of the
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GAS TRANSFER TO AND FROM AQUEOUS SOLUTIONS

Thomas R. Camp,[a] M. ASCE
(Proc. Paper 1701)

ABSTRACT

Reviews development of recent advances in the theory of gas transfer with discussion of various items. Emphasis is put on the need for more research on the subject.

Gas transfer to and from aqueous solution is of prime importance to sanitary engineers in water and liquid waste treatment and in studies of the oxygen balance in polluted streams. The gases of principal interest are oxygen, carbon dioxide and hydrogen sulfide. Some recent advances in the development of the theory of gas transfer have thrown new light on the subject and have opened the door to further research which should be very fruitful. It is the purpose of this paper to discuss these developments.

In 1924 Lewis and Whitman(1) advanced the two-film theory of the absorption of a gas by a liquid. The theory is based on the assumption of a gas film and a liquid film located at the interface through which the gas must pass by molecular diffusion and beyond which the concentration of the gas is uniform. For most problems dealt with by sanitary engineers, it has been found that the resistance of the gas film is negligible by comparison with that of the liquid film and that only the liquid film need be considered. The time rate of solution (or extraction) of the gas is as follows:

$$\frac{dc}{dt} = K_L a(c_s - c) \qquad (1)$$

in which c = the concentration of the dissolved gas in the body of the liquid at time t;

Note: Discussion open until December 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1701 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 4, July, 1958.

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c_s = the concentration of the dissolved gas at the gas-liquid interface (the saturation concentration);

a = the interfacial area per unit of volume of liquid;

and K_L = the liquid film coefficient

If the concentrations c and c_s in equation (1) are expressed in ppm, (parts per million) and the rate $\frac{dc}{dt}$ is in ppm per hr, $K_L a$ has the dimensions of hrs^{-1}. If a is in cm^2 per cm^3, K_L has the dimension of cm per hr. Many valuations have been made experimentally of the liquid film coefficient, K_L, for oxygen transfer from air bubbles rising through water. For a 2.4 mm diameter bubble rising through water at 15°C, Scouller and Watson[2] determined K_L at 140 cm per hr. Adeney and Becker[3] found variations from 32 to 230 cm per hr. Ippen[4] et al reported values of 7 to 295 cm per hr.

In 1956 Dobbins[5] presented an excellent theoretical discussion of the liquid film theory of gas transfer in which he shows that $K_L = \frac{D_L}{L}$ with an unbroken film where D_L is the molecular diffusion coefficient in the liquid film and L is the thickness of the film. With moving gas bubbles, however, shear is always present to continuously change the liquid film. Similarly in flowing streams and in tanks or reservoirs, motion of the liquid is always present in varying degrees to change the liquid film. Dobbins shows that where the film is being continuously replaced or renewed K_L is practically independent of L and is given by:

$$K_L = \sqrt{D_L r} \qquad (2)$$

in which r = the average rate of renewal of the liquid film.

In 1953 Pasveer[6] introduced the concept of mixing of the surface film with the main body of the liquid and derived an expression for the liquid film coefficient similar to equation (2). Pasveer started with Stefan's integration of Fick's law for still water and then differentiated it after introducing the rate of surface renewal. Pasveer's value for the liquid film coefficient is $\frac{2}{\sqrt{\pi}}$ or 1.13 times the value given by equation (2). The significant feature of Pasveer's work is that his value for the liquid film coefficient is substantially the same as that derived by Dobbins, even though his method of derivation was quite different.

In a study of non-isotropic turbulence, O'Connor and Dobbins[7] have shown that the rate of renewal of the surface film, r, is substantially equal to the velocity gradient in the liquid at the interface. The author of this paper has shown[8] that for rising gas bubbles:

$$K_L = \sqrt{m}\sqrt{D_L \frac{v}{d}} \qquad (3)$$

in which v = the bubble velocity with respect to the surrounding liquid,

d = the diameter of the bubbles,

and m = a proportionality constant

The experiments of Ippen[4] et al included depths of aeration diffusers ranging from 2.1 to about 12 ft. Fig. 1 shows the results of the experiments of Ippen et al with $\sqrt{m}$ plotted as a function of the bubble diameter. The numeral at each experimental point plotted on the figure represents the depth of the tank in feet. Also shown on the figure by means of x's are the experimental points in which the water contained a synthetic detergent. The concentration of the detergent in ppm is shown inside the block at each experimental point. It will be noted that the presence of detergents has a very marked effect in reducing the value of $\sqrt{m}$, and hence the value of the liquid film coefficient, K_L. The figure also indicates the effect of bubble size on the liquid film coefficients. The points on the figure, shown by means of triangles, are the results of experiments by Ippen et al with pure oxygen bubbles. The liquid film coefficient, K_L, for pure oxygen seems to be somewhat less than for air, although the results are not too conclusive. It should be noted that the transfer rate measured was for oxygen in both cases.

Experiments made under the author's direction on diffused aeration and carbonation of mixed municipal sewage and alkaline textile mill wastes indicated values for $\sqrt{m}$ for 2.5 mm bubbles ranging from 0.21 to 0.44 and averaging 0.31 for CO_2 in flue gas containing 2.6 to 7.5% CO_2, and ranging from 0.065 to 0.17 for oxygen in air with an average of about 0.115. In computing these values of $\sqrt{m}$, it was necessary to use values of D_L for pure water, about 1.9×10^{-5} cm^2 per sec for CO_2 and about 2.4×10^{-5} cm^2 per sec for O_2. These values of $\sqrt{m}$ are shown on Fig. 1 by means of squares for CO_2 and by means of crosses for oxygen. When these values are compared with the values determined in the Ippen experiments as shown in Fig. 1, it will be noted that they are very much lower. This probably indicates lower values of D_L because of impurities in the water.

In order to compute the liquid film coefficient, K_L, from equation (2), one must have reliable values for the coefficient of molecular diffusion, D_L, and of the rate of renewal of the surface film, r. Both of these must be evaluated experimentally. In order to make an experimental evaluation of r for a given condition, one must know the value of the molecular diffusion coefficient. The molecular diffusion coefficient, however, may be evaluated experimentally without knowledge of the rate of surface renewal.

The rate of diffusion per unit area according to the law of Fick is

$$c \frac{ds}{dt} = -D_L \frac{dc}{ds} \qquad (4)$$

where $c \frac{ds}{dt}$ is the quantity of particles per sec diffusing through 1 sq cm of a boundary in the direction of s,

$\frac{ds}{dt}$ is the diffusion velocity,

$-\frac{dc}{ds}$ is the concentration gradient, and

D_L is the diffusion coefficient.

For colloidal particles which are large compared with the size of water molecules, the water may be considered as a continuous medium and the law of Stokes used for the drag force. Einstein[9] has derived the value of the diffusion coefficient for this case as follows:

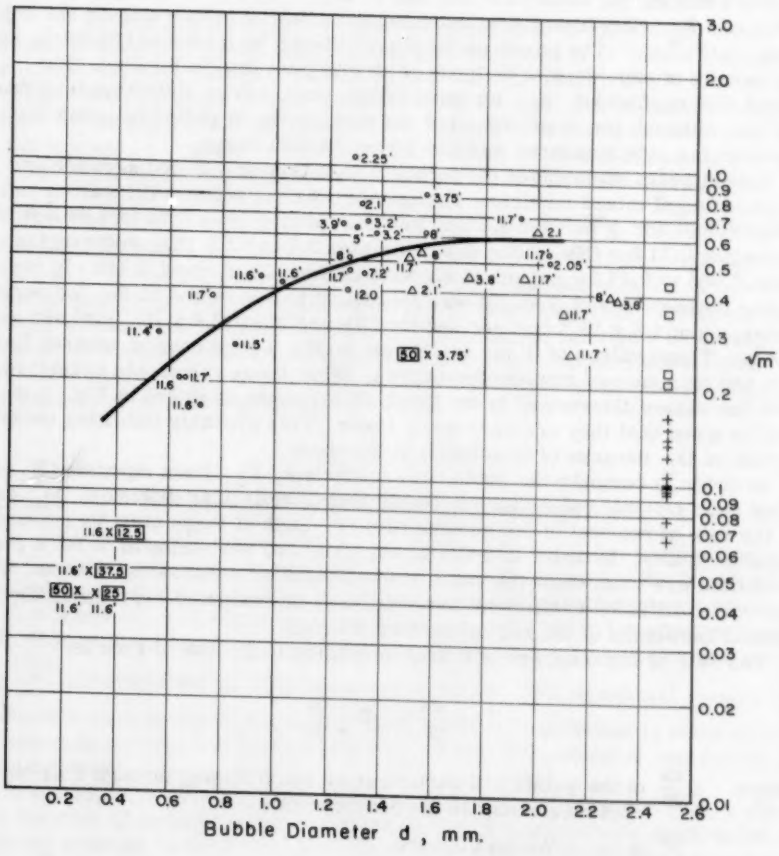
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
√m
0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6
Bubble Diameter d , mm.
50 X 3.75'
11.6 X 12.5
11.6' X 37.5
50 X X 25
11.6' 11.6'

Fig. 1

$$D_L = \frac{1}{3\pi d\mu} \frac{RT}{N} \quad (5)$$

where $3\pi d\mu$ is the Stokes drag for particles of diameter, d, in a fluid of viscosity, μ,

R is the gas constant,

T is the temperature in degrees K ($^{0}C + 273$), and

N is Avogadro's number.

The coefficient of diffusion for ions and small molecules is not adequately stated by equation (5), since the particles are of the same order of size as water molecules and the water cannot therefore be considered as a continuous medium. Equation (5), the Stokes-Einstein expression for the coefficient of diffusion of colloidal particles indicates that the value of the coefficient should vary directly with the value of T/μ. In the absence of experimental values for the coefficient of diffusion of a substance at temperatures other than the temperature at which experiments have been made, coefficients have been computed by means of the Stokes-Einstein relation.

Some measured values for diffusion coefficients of various substances in pure water, as reported in International Critical Tables and other sources, are given in Table 1.

It will be noted from Table 1 that the values available are at a single temperature in most instances. There is a dearth of information at other temperatures even for diffusion in pure water. The diffusion coefficient of dissolved oxygen in pure water is reported by International Critical Tables at 2.02×10^{-5} at $18^{0}C$ as shown in Table 1 and Fig. 2. Fig. 2 also shows the values of T/μ as a function of the temperature. In the absence of experimental values for the diffusion of dissolved oxygen at temperatures other than $18^{0}C$, the coefficients have been computed by means of the Stokes-Einstein relation and are shown by the solid line on Fig. 2. Also shown on Fig. 2 by means of a broken line are the relative effects of temperature on D_L for dissolved oxygen, based on experiments by Adeney.

The coefficients of diffusion in pure water for both carbon dioxide and 0.05 molal sodium chloride as taken from the International Critical Tables are shown on Fig. 2. It will be noted that the coefficient for sodium chloride conforms almost exactly to the Stokes-Einstein temperature relation, but that the coefficient for carbon dioxide does not. The decrease in D_L for chlorine with increase in temperature is puzzling. The values of the coefficient for hydrogen sulfide as shown on Fig. 2 were taken from two sources and are of questionable reliability for pure water. The values of the coefficient in pure water for sodium chloride and potassium chloride at one temperature, taken from the International Critical Tables, indicate that the coefficient is approximately constant at low salt concentrations but begins to increase in concentrations of about 6%.

There is almost a complete absence of information on diffusion coefficients for various substances in aqueous solutions and suspensions such as are encountered with natural and waste waters. The Stokes-Einstein relation, equation (5), tends to indicate that the effect of the impurities in the water on the value of the diffusion coefficient for a particular substance is felt through the viscosity of the medium. Inasmuch as it has now been demonstrated that

TABLE 1. DIFFUSION COEFFICIENTS IN PURE WATER

Solute	Molal Conc.	Temp. °C	D_L sq cm/sec
Dissolved Oxygen (O_2)	---	18	2.02×10^{-5}
Hydrochloric Acid (H^+ and Cl^-)	0.1	19	2.56×10^{-5}
Sodium Chloride (Na^+ and Cl^-)	0.1	15	1.09×10^{-5}
Calcium Chloride (Ca^{++} and Cl^-)	2.0	10	0.79×10^{-5}
Chlorine (Cl_2)	0.1	12	1.41×10^{-5}
Chlorine (Cl_2)	---	16	1.26×10^{-5}
Sulfuric Acid (H^+, HSO_4^- and SO_4^{--})	1.0	12	1.3×10^{-5}
Magnesium Sulfate (Mg^{++} and SO_4^{--})	1.0	7	0.35×10^{-5}
Ammonia (NH_3, NH_4OH, NH_4^+ and OH^-)	1.0	15	1.78×10^{-5}
Acetic Acid (CH_3COOH, H^+ and CH_3COO^-)	0.2	13.5	0.89×10^{-5}
Methyl Alcohol (CH_3OH)	---	18	1.37×10^{-5}
Glycerol ($C_3H_5OH_3$)	---	20	0.83×10^{-5}
Glucose ($C_6H_{12}O_6$)	---	18	0.57×10^{-5}
Urea (NH_2CONH_2)	---	20	1.18×10^{-5}
Various dyes	---	18	$(0.17 \text{ to } 0.58) \times 10^{-5}$
Various preteins	---	7 to 17	$(0.04 \text{ to } 0.13) \times 10^{-5}$
Gamboge particles (d = 0.5 microns)	---	20	0.85×10^{-8}

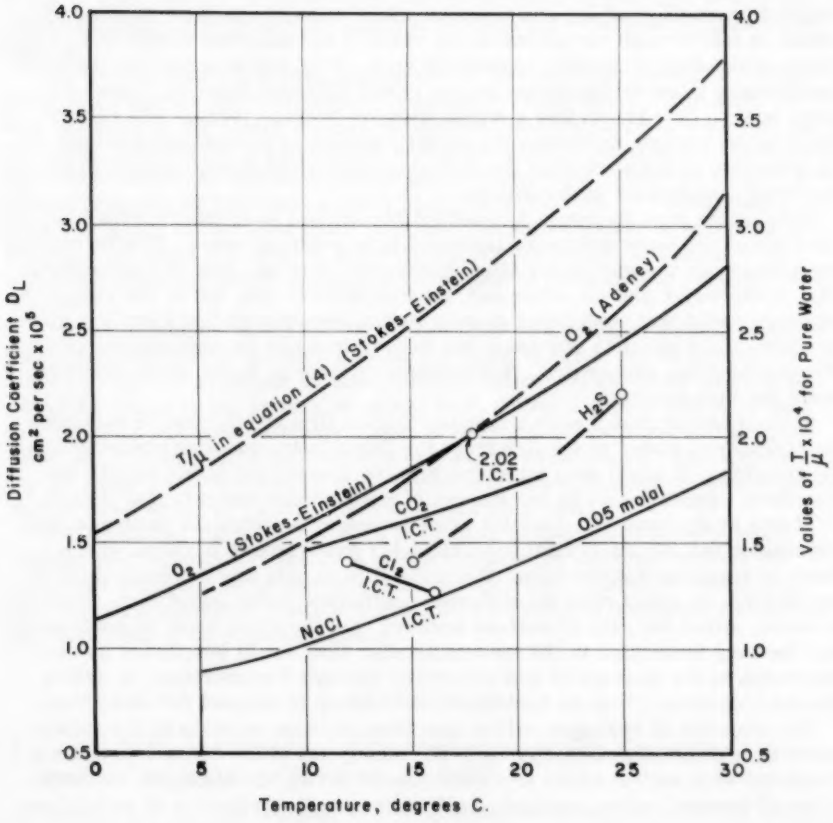

Fig. 2 Effect of temperature on diffusion coefficients in pure water.

the rate of absorption of gases in and extraction of gases from aqueous solutions depends upon the diffusion coefficient of the gases in a liquid film at the gas water interface, it is essential to have reliable values for the diffusion coefficient in the liquid film.

The experiments of Ippen and others on bubble diffusion indicate that the presence of surface active agents in the water greatly affects the value of the liquid film coefficient for gas transfer. It is essential to know whether this effect is felt through variations in the value of the diffusion coefficient, in the value of the rate of surface renewal or both. It is probable that the diffusion coefficients in many liquid wastes are vastly different from the values for pure water. It is known that a solute with low surface tension will concentrate in the surface and lower the surface tension of an aqueous solution. It is important to know whether the diffusion coefficient in the liquid film is markedly changed by such a solute.

During the past 30 years, a considerable amount of research work has been done with monomolecular layers of fatty acids on water. The layers are produced when the molecules align themselves with one end, the hydrophylic end, in the water and the other end, the hydrophobic end, out of the water. It has been found that such films greatly reduce the evaporation from the water surface. This effect is now being put to practical use in reducing evaporation losses from reservoirs. Hexadecanol appears to be the most effective agent for this purpose.

It would seem that if such a monomolecular film is effective in retarding the transfer of water molecules from the liquid to the gaseous phase across the interface, it might also interfere with the transfer of gases across the interface. Experiments by the Bureau of Reclamation indicate that it does.

If this is the case, the question arises whether equation (2) is an adequate expression for the liquid film coefficient for gas transfer in cases where there is a monomolecular film. Since the film is only one molecule thick, it should have no effect upon the diffusion coefficient in the liquid film. It might, however, affect the rate of surface renewal. On the other hand, it would seem that the long molecules in the monomolecular film would simply act as obstructions to the passage of gas molecules through the interface. If this is the case, equation (2) is an inadequate expression to account for this effect.

The transfer of hydrogen sulfide gas from aqueous solution to the atmosphere is of interest principally from the standpoint of the odors it produces. Problems with sulfide odors are encountered during the anaerobic decomposition of organic matter containing sulfur, during the reduction of sulfates to sulfides in anaerobic decomposition of organic matter and during the reduction of pH of alkaline trade wastes containing sodium sulfide. Sodium sulfide is widely used in the textile industry, in tanneries and in pulp mills.

In most industrial uses of sodium sulfide, lime or lye is also used to produce an extremely high pH of the liquor. When sodium sulfide is dissolved in water, it ionizes into sodium and sulfide ions. The sulfide ions immediately react with the hydrogen ions from the water to produce hydrosulfide ions. This reaction releases hydroxide ions, thus increasing the pH value of the solution. Sodium sulfide is therefore a strong alkali, analagous to sodium carbonate.

A study of the ionization constants of hydrogen sulfide and hydrosulfide indicates that there is a negligible amount of sulfide ion present, even at pH values of 12. Practically all of the sulfides present between pH values of 10 to 12 are in the form of hydrosulfide ion. As the pH is lowered below 10,

there is an increasing amount of hydrogen sulfide gas in solution such that at pH 4, 99.9% of the sulfides present are in the form of hydrogen sulfide gas. Table 2 shows the effect of pH value on the per cent of the total sulfides present in the form of hydrogen sulfide gas. It will be noted that at pH 7 approximately half of the sulfides are in the form of hydrogen sulfide gas, whereas at pH 9, less than 1% of the sulfides are in the form of hydrogen sulfide gas. These relations are extremely important in dealing with liquors containing sulfides.

The rate at which hydrogen sulfide gas is liberated from water to the atmosphere depends upon the degree of agitation of the water. The more turbulent the mixing, the faster the gas is liberated and the more intense will be the odor. Laboratory studies made under the author's direction indicate that the concentration of hydrogen sulfide gas in streams of moderate turbulence should probably be less than about 1 ppm to avoid odors. To avoid odors from the spray of trickling filter nozzles, the concentration should probably be less than about 0.5 ppm. These figures are very approximate and considerable research under controlled conditions is desirable to confirm their reliability.

Table 2 indicates that if the pH of the water in a polluted stream is 7, the total concentration of sulfides may be 2.15 ppm in order not to exceed the limit of 1 ppm of hydrogen sulfide gas in solution. On the other hand, if the pH of the water in the stream is raised to 9, the total concentration of sulfides may be 116 ppm with only 1 ppm in the form of hydrogen sulfide gas. Thus, an increase in the pH value is one way to avoid odors. An increase in the pH value from 7 to 9 is in this respect equivalent to dilution with 54 parts sulfide-free water to 1 part of sulfide-containing water.

In conclusion, the author wishes to emphasize the importance of more fundamental research in the field of gas transfer as this field affects the problems encountered in sanitary engineering. Such research should yield results of considerable value in the solution of difficult problems and should pay dividends in an economic sense.

REFERENCES

1. Lewis, W.K., and Whitman, W.C., "Principles of Gas Absorption," Industrial and Engineering Chemistry, 16 (1924).

2. Scouller, W.D., and Watson, W., "The Solution of Oxygen from Air Bubbles," The Surveyor and Municipal and County Engineer, (London) 86, No. 2215, 15 (1934).

3. Adeney, W.E., and Becker, H.G., "The Determination of the Rate of Solution of Atmospheric Nitrogen and Oxygen by Water," Phil. Mag., 38, 317 (1919) and 39, 385 (1920).

4. Ippen, A.T., Campbell, L.G., and Carver, C.E., Technical Report No. 7, May, 1952, and Ippen, A.T., and Carver, C.E., Technical Report No. 14, April, 1955, Hydrodynamics Laboratory, Mass. Inst. of Tech., Cambridge, Mass.

5. Dobbins, W.E., "The Nature of the Oxygen Transfer Coefficient in Aeration Systems," Biological Treatment of Sewage and Industrial Wastes, Reinhold Publishing Corp., 1956, p. 141.

TABLE 2. IONIZATION OF HYDROGEN SULFIDE IN AQUEOUS SOLUTION AT 25°C

pH	Per Cent H_2S
4	99.9
5	98.9
6	89.7
7	46.5
7.5	21.6
8	8.0
8.5	2.68
9	0.86
10	0.087

6. Pasveer, A., "A Study of the Aeration of Water," Sewage and Industrial Wastes, November, 1953, p. 1253.

7. O'Connor, D.J., and Dobbins, W.E., "The Mechanism of Reaeration in Natural Streams," Jour. San. Eng. Div. American Soc. C. E., Dec., 1956.

8. Camp, T.R., Discussion of Paper by O'Connor and Dobbins, Jour. San. Eng. Div. American Soc. C. E., April, 1957.

9. Einstein, A., Ann. Physik (4) (17, 549 (1905); 19, 371 (1906).

Paper 1702 July, 1958 SA 4

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

EFFECTS OF MECHANICS OF FLOW ON CORROSION

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(Proc. Paper 1702)

ABSTRACT

Conflicting corrosion data has shown velocity of fluid flow under many conditions to be inadequate as a corrosion parameter. This paper presents fundamental hydraulic concepts as applied to corrosion and the rate of transfer of interacting chemicals in flow over plates, rotating disks and through pipes.

INTRODUCTION

Velocity of water and its effects on the corrosion of metals have been studied empirically by many investigators. However, velocity by itself is an indefinite parameter of flow conditions within pipes of various sizes and surface characteristics. Other factors such as boundary layers, mechanisms of oxygen transfer, interfacial films of inhibitors and passivators all play an important part. From the preponderance of conflicting data concerning corrosion and the effects of various inhibitors it is evident that greater consideration must be given to the fundamentals involved. It is the object of this presentation to show by theoretical considerations and experimental data that the effects of velocity on corrosion are most constructively realized when approached from the basic concepts of fluid mechanics.

Great strides in corrosion research through electrochemical studies over the past half century have not been paralleled by equally significant studies on the tremendously important physical factors which control mixing and the transport of interacting substances in the corrosive medium, which in turn

Note: Discussion open until December 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1702 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 4, July, 1958.

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influence corrosion reactions. These physical factors may be interpreted in relation to the mechanics of fluid flow. It will be seen from the studies presented herein that differences, and at times apparent contradictions, in corrosion results are often due to variations in flow conditions, which are in turn caused by differences in methods of testing employed.

Theory of Corrosion

Most metals, such as the iron used in structural materials, do not occur in their pure form in nature. Metallurgical processes separate the metals from their ores by the use of energy from thermal, chemical, or electrical reactions. The products thus obtained are in a higher state of energy than before and tend to revert back to their primitive states in a suitable environment.

Studies over the past half century have shown that corrosion in an aqueous solution is an electrochemical process in which the rate of corrosion is a function of the rate of current flow between anodes and cathodes on the surface of the metal. The primary electrolytic reactions in the corrosion process, as well as the secondary reactions involving oxygen, have been presented and discussed in these Proceedings by Camp[1] and need not be repeated here.

Oxygen plays an important role as a depolarizer in corrosion by natural water. When corrosion products form on the metal surface, however, the effect of oxygen becomes more complex than when bare metal is corroding, because the concentrations of oxygen present can influence the characteristics of the corrosion product film. Cox and Roetheli[2] found in short-term tests that the rate of corrosion of steel was approximately proportional to the oxygen concentration up to 5.5 c.c. per liter. At higher oxygen concentrations the corrosion products were composed predominantly of red gelatinous hydrated ferric oxide, while at lower oxygen concentrations a layer of porous black magnetic oxide of iron formed on the metal surface. In a recent investigation by Uhlig, et al.[3] of the corrosion of iron by natural waters containing dissolved salts, a linear relationship was found between the corrosion rate and dissolved oxygen concentrations up to oxygen concentrations corresponding to air saturation or above.

Since oxygen plays such an important role in corrosion, the manner in which it is brought to the walls of the pipe must be considered. The effect of the velocity of water on the transfer of oxygen and other processes involved in corrosion reactions should be evaluated on the basis of certain hydraulic concepts, which are developed in this paper.

Previous Work

Friend[4,5] investigated the effects of velocity on corrosion rates as determined by weight losses of iron. In natural water the corrosion rate decreased with increasing velocity, and at a high velocity of 8.0 ft./sec. no weight losses were observed. In acid solutions, however, Friend and Dennet[6] found that the corrosion rate increased with increasing velocities. In 1923 Speller and Kendall[7] published a paper which contradicted the results of Friend. They found that in natural water the corrosion rate was low in laminar flow, increased rapidly in the transition range, and continued to increase, but at a

lower rate, in the range of turbulent flow in steel pipes.

Many investigators have noted the interrelationship between oxygen and velocities in their effects upon corrosion. Whitman[8] wrote in 1925 that the effect of higher velocities should be to increase the corrosion rate due to increased diffusion of oxygen and the breaking down of protective films on the metal surfaces. Wilson[9] reported that velocity had an important effect in governing the thickness of the film through which oxygen must diffuse in the corrosion process. Evans[10] stated in 1925 that at high velocities corrosion is not likely to begin as soon on a previously uncorroded surface, because the greater turbulence results in more uniform oxygen concentrations on the metal.

Roetheli and Brown,[11] who rotated steel specimens in oxygenated Cambridge, Massachusetts, tap water, found that as the rotational velocities increased, corrosion increased to a maximum value, decreased to a low rate, and increased again to a somewhat higher rate at very high velocities. These variations in the corrosion rate were attributed to three factors: (a) differences in the type and uniformity of the corrosion product film, (b) changes in film thickness, and (c) erosion. At low velocities the corrosion products consisted mainly of granular magnetic oxide of iron, while at high velocities a predominance of hydrated ferric oxide was attributed to the greater supply of oxygen. A maximum corrosion rate was reached at some intermediate velocity in the presence of two opposing tendencies—an accelerated transfer of oxygen due to a reduction in liquid film thicknesses, and increased rate of formation of ferric hydroxide by oxidation of ferrous ions. At extremely high velocities the nonuniform mechanical removal of the ferric hydroxide film increased the corrosion rate.

In 1938 Frese[12] showed that with high oxygen concentrations the corrosion rate may drop to low values as the iron tends to become passive. In the same year Speller[13] reported that the effect of velocity was essentially to bring oxygen into more intimate contact with cathodic surfaces. Active oxidation taking place at high velocities indicated the formation of protective ferric hydroxide film, and higher velocities tended to keep this formation of protective film nearer to the reaction points, thus stifling the corrosion reaction.

Wormwell[14] reported that corrosion rates were determined largely by the rate of oxygen supply to the metal surfaces. Hatch and Rice[15] showed that the effect of velocity was to change the oxygen concentration gradient near the metal surface. As the velocity of flow continued to increase, the renewal of the liquid layer near the metal surface became sufficient to cause little change in oxygen concentration in spite of the amount of oxygen used up in the corrosion reactions. At very high velocities the oxygen concentration at the metal surface was only slightly greater than at moderate velocities, and thus the corrosion rate increased but a small amount.

Copson,[16] in a review of the literature on the effects of velocity on corrosion by water, found that velocity was a major factor, but that most of the available information on the subject was somewhat contradictory. In general he found that corrosion increased with increase in velocity, but the effect was sometimes the opposite.

In 1956 Streicher[17] presented the results of tests using 2 x 8 inch strips of various metals immersed in Colorado River water. He explained the drop in corrosion rates of steel at higher velocities as due to passivation brought about by the increased oxygen supply. Uhlig[18] has reported extensively on

the effect of oxygen passivation at high oxygen concentration.

The vast amount of published data on corrosion, only a small amount of which is reviewed here, shows clearly that there is a distinct relationship between physical and chemical conditions in the corrosive medium. The velocity of the water, degree of turbulence, geometry of pipes, and other physical factors have a direct influence on the movement of dissolved oxygen and ions throughout the fluid. These factors in turn exert effects on the formation of corrosion products, depolarization, and thus on the corrosion reaction itself.

Theoretical Approach to the Effect of Velocity

Many authors, when dealing with research concerning the effect of movement of corrosive media upon corrosion rates, consider the velocity of such media relative to the test specimen as the only variable. It will be shown later that velocity by itself can be considered as the only variable only in similar types of flow, and then only to a certain extent, since the criterion of hydraulic similarity is not the same for all corrosion phenomena. To be more specific, it is necessary to discuss flow in which not only velocity alone is taken into account, but in which other circumstances such as boundary conditions; flow on plates, disks, cylinders and pipes; steady or unsteady state of flow; and laminar or turbulent conditions are considered. It can be predicted at the beginning that different flow conditions at the same velocities will produce very different test results. This will be shown by theoretical considerations and confirmed experimentally.

The role played by oxygen in the corrosion process is such that in many cases its presence is the determining factor in the entire corrosion reaction. Since flow conditions affect the availability of oxygen, the principal external chemical taking place in the corrosion reaction, much of the emphasis in this presentation will be to show the effect of flow conditions on the availability and transfer of dissolved oxygen.

Purely mechanical effects of velocity such as erosion or cavitation will not be discussed in this study. These effects are generally very localized and are seldom encountered in water lines in which velocities are not high.

Theoretical and experimental considerations presented herein are based on the premise that the metal surfaces are fully exposed to the corrosive media at the beginning, without benefit of artificially applied protective coatings, and that coatings of corrosion products will form naturally in time according to the environmental and flow conditions.

In these studies the corrosion rate is taken as a function of the rate at which oxygen diffuses toward the metal surface in the oxygen concentration range found in natural water supplies. Diffusion of solutions of gases in water is a very slow process. Fick's law of diffusion states:

$$\frac{dW}{dt} = K_d A \frac{dc}{dy} \qquad (1)$$

where:

W = weight of dissolved substance

t = time

A = cross-sectional area

c = concentration

y = distance

K_d = coefficient of diffusion

Diffusion of gases is a result of molecular action. The coefficient, K_d, is variable with different substances and is a function of the temperature. In many respects it is comparable to the kinematic viscosity, ν, which measures the diffusion of energy in a fluid.

Although the experimental work reported herein was conducted in steel pipes, many other types of flow are encountered in various corrosion tests. Three of the most common types of corrosion tests involve the use of flat plates, rotating discs, and pipes. The flow conditions in these tests are briefly reviewed to indicate their influence on test results.

1. Flow Over a Thin Flat Plate

It is known that the particles of a real fluid in contact with a boundary have the same velocity as the boundary itself. In other words, the fluid adheres to the walls confining the flow, and frictional forces retard the motion of the fluid in a generally thin layer near the walls. In this layer the local velocity of the fluid increases from zero at the wall to its full value in the undisturbed flow as shown in Fig. 1.

The rate of change of the velocity near the wall follows a definite law. The zone near the wall is known as the boundary layer. The boundary layer thickness is considered as the distance normal to the wall where the local velocity is 99 per cent of the undisturbed velocity.

The local shearing stress, τ, is given by the equation:

$$\tau = \mu \frac{du}{dy} \tag{2}$$

where:

μ = dynamic viscosity of the fluid, and

$\frac{du}{dy}$ = velocity gradient along the wall

(See Fig. 1). The velocity gradient is very large near the wall, giving rise to high shearing stresses. However, as the local velocity approaches the undisturbed velocity asymptotically the local shear stresses become smaller and smaller, and eventually become zero.

As the fluid reaches the upstream edge of a plate, fluid particles may adhere to the plate. Shearing stresses are developed, and through the action of momentum transfer more and more fluid particles are slowed down and the boundary layer is developed. The flow within the boundary layer at the leading edge of the plate is laminar. Beyond this the flow becomes turbulent according to the Reynolds number:

$$R_x = \frac{Ux}{\nu} \tag{3}$$

where:

U = undisturbed velocity

x = distance from the leading edge, and

ν = kinematic viscosity of the fluid

The value of the Reynolds number at which the flow becomes turbulent is approximately 5×10^5. The transition from laminar to turbulent flow begins at approximately $R_x = 3 \times 10^5$.

The thickness of the laminar boundary layer is given by the Blasius relation as:

$$\delta = \frac{5x}{R_x} \tag{4}$$

The thickness of the turbulent boundary layer is also given by the Blasius relation:

$$\delta = \frac{0.38\,x}{R_x^{1/5}} \tag{5}$$

Confined within the turbulent boundary layer is a zone termed the laminar sub-layer. The existence of this very thin layer next to the wall stems from the fact that at the wall the flow must be laminar. Since the logarithmic velocity distribution curves within the boundary layer are not satisfied adjacent to the wall, the velocity distribution is therefore assumed to be linear. The zone of the laminar sub-layer is confined within the limits of the linear velocity distribution and has been empirically determined as:

$$\delta = 11.6 \frac{\nu}{\sqrt{\frac{\tau_o}{\rho}}} \tag{6}$$

The importance of the laminar sub-layer lies in the fact that as long as the protuberances at the wall surface are contained within this layer the boundary is hydraulically smooth, and the friction coefficient is independent of the relative roughness value. If the protuberances are of a higher value than the laminar sublayer thickness, the laminar layer is disrupted, and the relative roughness affects the friction coefficient.

2. Rotating Disk in an Infinite Fluid

When a smooth flat disk rotates about an axis perpendicular to its plane, with a uniform angular velocity of ω in a liquid otherwise at rest, the fluid layer near the disk is carried by it through friction and is thrown outwards owing to the action of centrifugal forces. This action is compensated for by particles which flow in an axial direction towards the disk, to be in turn carried and ejected centrifugally.

As in the case of a flat plate, a boundary layer is also formed with a rotating disk. The flow in this layer may be either laminar or turbulent, with the transition occurring at Reynolds number of approximately 3×10^5. The

representative Reynolds number is given in this case by the equation:

$$R_D = \frac{\omega R^2}{\nu} \tag{7}$$

where:

ω = angular velocity

R = radius of the disk, and

ν = kinematic viscosity

The thickness of the boundary layer and the flow produced by the centrifugal action may be expressed as follows for laminar and turbulent flows:

a) Laminar Flow

$$\delta = \frac{2.58r}{\sqrt{R_r}} \tag{8}$$

$$q_d = 0.886\pi R^3 \omega R_D^{-1/2} \tag{9}$$

b) Turbulent Flow

$$\delta = \frac{0.526r}{R_r^{1/5}} \tag{10}$$

$$q_d = 0.219\pi R^3 \omega R_D^{-1/5} \tag{11}$$

where:

δ = thickness of the boundary layer

r = radial distance to the point considered

R_r = Reynolds number at this point = $\frac{\omega r^2}{\nu}$

R = radius of the disk

ω = angular velocity

R_D = Reynolds number of the disk

q_d = discharge of the disk

3. Flow Through Pipes

The transition from laminar flow to turbulent flow in pipes occurs at a Reynolds number of:

$$R = \frac{VD}{\nu} = 2{,}000 \tag{12}$$

where:

V = mean velocity, and

D = diameter

The laminar range is seldom found in pipe flow with the possible exception of flow at the entrance of a pipe line leading from a reservoir.

The majority of pipe flow problems deal with steady, uniform, fully developed flow. To be sure, there will still be a thin layer of laminar flow next to the boundary if the boundary is smooth. The laminar sub-layer, however, could hardly exist at the boundary if the roughness magnitude, k, is greater than the value of δ'.

In fully developed turbulent flow the limit of boundary layer growth is the pipe centerline. Once steady uniform flow is developed, the Reynolds number remains constant as long as the pipe cross-section remains constant.

The Karman-Prandtl equation for velocity distribution in turbulent flow near smooth boundaries is given by:

$$\frac{v}{\sqrt{\tau_0/\rho}} = 5.75 \log_{10} \frac{\sqrt{\tau_0/\rho}\ y}{\nu} + 5.5 \qquad (13)$$

For turbulent flow near rough boundaries the Karman-Prandtl equation is as follows:

$$\frac{v}{\sqrt{\tau_0/\rho}} = 5.75 \log_{10} \frac{y}{k} + 8.5 \qquad (14)$$

where:

v = local velocity

y = distance normal to the boundary

k = magnitude of roughness

ρ = mass density

τ_0 = intensity of surface shear = $\frac{f}{4} \rho \frac{V^2}{2}$

where:

V = mean velocity

f = Darcy-Weisbach friction coefficient

The term $\sqrt{\frac{\tau_0}{\rho}}$ has the dimension of velocity and is known as the shear velocity.

Introducing the mean velocity and combining the equations above results in the following relationship for velocity distribution:

$$\frac{v - V}{V\sqrt{f}} = 2 \log_{10} \frac{y}{r} + 1.32 \qquad (16)$$

where r is the radius of the pipe. The above equation indicates that the relative velocity distribution in a circular pipe is a function of $\frac{y}{r}$ only for a given value of f regardless of whether the boundary is rough or smooth.

Theoretical Influence of Flow Conditions Upon Corrosion Rates

The three cases which will be considered, as before, will be flat plates, rotating disks, and flow through pipe lines. It has been shown by other investigators that corrosion proceeds at approximately the rate at which oxygen is supplied to cathodic areas. As corrosion proceeds, the water in contact with the metal surface becomes depleted in oxygen and charged with corrosion products. Concentration gradients form, and oxygen flows toward the metal, while metal ions and dissolved compounds move from the metal toward the main body of the liquid.

1. Flow Over a Thin Flat Plate

Certain authors have considered oxygen transfer in a corrosive medium as simple diffusion of oxygen through the boundary layer. It does not appear that this approach represents the true condition. Diffusion, especially of gases in water, is a very slow process, the equation of which has been given previously. If the concentration of oxygen in the main flow is designated as c_s and that at the metal surface as c_m, the concentration gradient becomes $\frac{c_s - c_m}{\delta}$, where δ is the boundary layer thickness.

The oxygen transfer in the case of flat plates is not strict diffusion, because there is flow in the boundary layer itself. Thus this flow in the boundary layer brings new oxygen to the metal surface and takes away some corrosion products.

As indicated in Fig. 2, the velocity, U, at the leading edge of a plate is uniform. The oxygen concentration, c_s, also is uniform. At x distance downstream, the shear stress on the boundary decelerates the fluid in the boundary layer zone, δ_x. The laminae then has a loss of momentum of:

$$dM = \rho u (U - u) \int y \tag{17}$$

and the total loss of momentum of the fluid in the boundary layer at x distance from the leading edge becomes:

$$\Delta M = \rho \int_0^{\delta_x} u (U - u)\, dy \tag{18}$$

In the development of the boundary layer theory by Schlichting,[19] this equation has been integrated as follows:

$$\Delta M = \rho \Theta U^2 = \rho \int_0^{\delta_x} u (U - u)\, dy \tag{19}$$

The parameter, θ, whose dimension is a length, is called the momentum thickness, and is associated with the total moment deficit in the boundary layer. The term θ has been evaluated numerically[20] for a flat plate at zero incidence as:

$$\Theta = 0.664 \sqrt{\frac{\nu x}{U}} \tag{20}$$

Replacing this value in the equation above results in the expression:

$$\Delta M = CU^{1.5} \tag{21}$$

The loss of momentum of a fluid flowing over a flat plate can be compared with the loss of dissolved oxygen due to corrosion. They are similar in that while moment is lost by friction at the walls of a plate, oxygen is depleted by reactions at the plate surface also. Moment is transferred by the faster fluid particles by shear effect due to the velocity gradient $\tau = u \frac{\partial u}{\partial y}$. Oxygen is also transferred by a gradient, which in this case is a concentration gradient, $W = k_d \frac{\partial c}{\partial y}$.

It can be deduced that there is a concentration boundary layer in the corrosion process, which will follow laws similar to those of the fluid boundary layer. This is especially true before a thick and semi-impermeable corrosion product layer becomes formed. At the laminae dy, the concentration deficit will be:

$$\Delta c = \varrho u(c_s - c_x) \tag{22}$$

The parameter, θ, may be expressed as:

$$\theta = \int_0^\delta \frac{u}{U} \left(1 - \frac{u}{U}\right) dy \tag{23}$$

In the case of the oxygen concentration concept, the relationship becomes:

$$\theta_c = \varrho \int_0^\delta \frac{u}{U} \frac{c_s}{U} - \frac{c_x}{U} \, dy \tag{24}$$

If the c_x distribution is similar to u, and since $\frac{c_s}{U}$ is constant, the following equation results:

$$\theta_c = \varrho \int_0^\delta \frac{u}{U} (A - \beta \frac{u}{U}) \, dy \tag{25}$$

in which β is also a constant.

It is apparent that the loss of oxygen will follow laws similar to those of loss of momentum, and thus:

$$\Delta_0 \propto U^{1.5} \tag{26}$$

If the loss is proportional to the total corrosion, the equation becomes:

$$h \propto U^{1.5} \tag{27}$$

in which h is the corrosion rate. This indicates that if the corrosion rate at one velocity condition is known, the rate at another velocity would be

proportional to the ratio of the velocities raised to the 1.5 power.

Streicher's results[17] on corrosion of black iron at velocities of 1 to 4 ft./sec. appear to be proportional to the velocities raised to the first power. The difference in the exponents can be explained by the fact that Streicher's corrosion rates were taken as the average corrosion over two weeks. During this time the corrosion products, with their protective effect, reduce the corrosion rate, since it is known that the average corrosion rate generally decreases with time. If oxygen diffusion is assumed through the boundary layer, the corrosion rates become proportional to the velocities raised to approximately the 0.8 power.

2. Rotating Disks

It has already been pointed out that when a disk is rotated a layer of liquid is carried with the disk and flows outward. Since the diffusion of oxygen is only a consequence of the depletion in the layers of fluid near the walls, the amount of oxygen that the induced flow will carry over the disk, or in other words, the relative importance of this flow, will determine whether oxygen will reach the disk by diffusion or by convection with the flowing water.

In general the disk rotates at high speed. The outward flow is considerable, and thus the amount of oxygen made available from it will be sufficient to reduce diffusion to a very low value or to zero. By replacing the term R_D in the discharge equation given previously by its value of $\frac{R^2\omega}{\nu}$, the resulting equation for turbulent flow becomes:

$$q_d = 0.219\,\nu^{0.2}\,R^{2.6}\,\pi\,\omega^{0.8} \qquad (28)$$

If it is assumed that the corrosion rate is proportional to the oxygen carried by the flow, corrosion will then be proportional to the angular velocity raised to the 0.8 power.

Friend,[6] in his study of corrosion in dilute sulfuric acid, reported that the corrosion rate was proportional to the angular velocity, ω. He stated that this was an approximation derived from experimental results. However, the experimental points were not presented in his curves, and therefore it is impossible to determine whether an angular velocity exponent of 0.8 or 1.0 best fits the dispersion of points. Even if the experimental points were to follow a trend closer to the angular velocity to the exponent of 1.0, the difference between these corrosion values and those predicted by theory would not be large.

3. Flow in Pipes

This study will be confined to fully developed flow in pipe lines, which usually is attained at a distance of 40 to 100 diameters from the pipe entrance or from pipe fittings. It is important to note the difference between the concept of a plate in an infinite fluid and a pipe through which the corrosive fluid flows. In the first case the boundary layer continues to grow with the movement of fluid, and the amount of oxygen available is almost unlimited. However, in the second case a particle of fluid enters the pipe with a determined amount of dissolved oxygen, and when this oxygen is depleted due to the corrosion reactions, there is no possibility of replenishing the supply. Laminar flow is very seldom found in water supply installations, and thus only the

case of turbulent flow will be discussed. (See Fig. 3)

The laminar sub-layer in a smooth pipe, although known to exist by theoretical considerations, is very difficult to observe experimentally. Flow exists in the laminar sub-layer, but it is very small in amount. The thinness of this layer and the small quantity of flow indicate the probability that the liquid in this layer may be depleted of oxygen when the state of fully developed flow is reached. Therefore oxygen will diffuse to the laminar sub-layer from the outer fluid zones, where there are higher concentrations of this dissolved gas.

Adjacent to the laminar sub-layer is a transition zone, above which fully turbulent flow occurs. In analyzing fully turbulent flow in pipe lines, we may assume that the mixing effect in the turbulent and transition zones destroys all gradients and makes the flow appear to be an oxygen-saturated core flowing within a layer of oxygen depleted liquid. Oxygen diffusion will take place through the thickness, δ', of the laminar sub-layer, which is given by the expression:

$$\delta' = \frac{11.6\,\nu}{\sqrt{\tau_s/\rho}} \qquad (29)$$

in which:

δ' = thickness of laminar sub-layer

ν = kinematic viscosity, and

$\sqrt{\tau_s/\rho}$ = friction velocity

The term $\sqrt{\tau_s/\rho}$ is equal to $V\sqrt{\frac{f}{8}}$, in which:

V = mean velocity = $\frac{Q}{A}$, and

f = friction factor

In smooth pipes in which the Reynolds number is between 2,000 and 100,000, the friction factor is expressed by:

$$f = \frac{0.316}{R_D^{1/4}} \qquad (30)$$

Assuming a linear gradient of oxygen in the laminar sub-layer, as was done for the velocity distribution in that layer, the concentration gradient can be expressed by:

$$\frac{dc}{dy} = \text{constant} = \frac{c_s - c_m}{\delta'} = \frac{\Delta c}{\delta'} \qquad (31)$$

in which $\Delta c = c_s - c_m$. Fick's law of diffusion can be expressed as:

$$\frac{dW}{dt} = K_d \frac{\Delta c}{\delta'} \quad \text{per unit area} \qquad (32)$$

Considering the corrosion rate, h, as proportional to the oxygen supply, this rate is expressed by:

$$h = CK_d \frac{\Delta c}{\delta'} \tag{33}$$

in which C is a dimensional constant. Replacing the term δ' by its expression in the above equation results in:

$$h = K_1 \frac{V}{R_D^{1/8}} \tag{34}$$

The preceding formula can be finally expressed as:

$$h = K_2 \frac{V^{0.875}}{D^{0.125}} \tag{35}$$

In the above equations the coefficients, K_1 and K_2, include the proportionality coefficient C, all numerical coefficients, and other factors which remain constant when only velocity and diameter of pipe are the variables.

From the preceding formulas, we may deduce the following:

1. In two flows with the same Reynolds number, the corrosion rates will be proportional to the velocity.
2. In pipes of the same diameter, corrosion will be proportional to the velocities raised to the 0.875 power.
3. In two flows of the same velocity the corrosion rate will be inversely proportional to the diameters of the pipes raised to the 0.125 power.

The preceding deductions have been based upon the existence of a laminar sub-layer. It was pointed out that the roughness of pipe walls can easily disrupt this layer. The formation of corrosion product layers increases the roughness. The disappearance of the laminar sub-layer was shown in the reported laboratory studies by both friction loss measurements and visual observations of the pipe specimens after several hours of corrosion.

If the laminar sub-layer disappears or becomes disrupted, the oxygen supply to the metal surface will be greatly changed. Rouse[20] states that the intensity of mixing across a zone in which the concentration of matter in solution varies is given by:

$$M_{turb.} = -\beta\epsilon\frac{dc}{dy} \tag{36}$$

where:

M = rate of matter transport, and

β = a proportionality coefficient

The negative sign indicates that the transport of matter is invariably in the direction of decreasing concentration.

The term ϵ is the kinematic eddy viscosity, and it is related to the mixing length, l, and the velocity gradient by the expression:

$$\epsilon = l^2 \frac{du}{dy} \quad (37)$$

It is seen that the rate of transfer of matter, such as oxygen in this case, will depend upon the velocity distribution and also upon the mixing length.

Nikuradse,[21] in a study of the variation of ϵ in smooth pipes in flow ranges above $R = 10^5$, found that ϵ is independent of Reynolds number. From this analysis, and from the preceding presentation concerning the existence of a laminar sub-layer, it can be deduced that two flows which are hydraulically similar may not necessarily be similar from the corrosion point of view. In the case of turbulent flow in rough pipes, corrosion will depend also upon the velocity, increasing as the velocity increases. Roughness, which affects the velocity distribution, will probably increase the value of ϵ, and thus of the rate of oxygen transfer.

Experimental Methods

From the preceding analyses it can be seen that flow conditions have a pronounced effect on the resulting corrosion rates. Therefore, in corrosion testing it is very important, in order to obtain consistent and useful results, that the test model approximate the conditions to be found in the field. In addition, since so many variables affect corrosion, the test apparatus must be designed in such a way as to allow the investigator to evaluate and control these variables.

A flow diagram of the pipeline network used in these studies is shown in Fig. 4. A photograph of the actual equipment is seen in Fig. 5. Each of the four identical pipeline systems was maintained at constant temperature. Stainless steel centrifugal pumps were employed for recirculation. The discharge from the pumps passed first through a section of pyrex tubing to stabilize the flow before coming into contact with the test specimens.

The rate of flow through the systems was measured continuously by means of pyrex and stainless steel rotameters. Low capacity pumps were used to feed fresh water into the reservoirs continuously in order to replenish the corrosive medium and to prevent excessive amounts of corrosion products in solution. Excess water was allowed to overflow from the reservoirs to waste.

Synthetic tap water was used in these studies. It was prepared by adding chemicals to demineralized water to form an "average" water, that is, a water of chemical composition comparable to that of many supplies in the United States. The chemical characteristics of the water were as follows:

Component	Concentration (ppm)
Ca	22
Mg	7
Na	26
SO_4	47
HCO_3	70
Cl	25

The water had a hardness of 84 ppm as calcium carbonate and a total alkalinity of 57 ppm. The pH after aeration was 8.05, and the pH_s was 8.28. The water was chlorinated daily. During the course of the experiments the pH

value of the water was maintained constant by the addition of dilute hydrochloric acid or sodium hydroxide.

The specimens were sections of steel pipe approximately 5 inches long. They were subjected to standard cleaning procedures employed in the Corrosion Laboratory of the Massachusetts Institute of Technology. After weighing, they were assembled into the systems by means of rubber hoses and hose clamps. Lucite washers placed between the specimens prevented galvanic or stray current effects. After removal from the systems, visual observations were made of the corrosion products, and the specimens were cleaned and weighed.

The temperature of the water was maintained at 20^{0} C., and it was constantly aerated. Dissolved oxygen determinations were made daily; and hardness, alkalinity, and iron were determined weekly.

Experimental Results

1. Laminar Flow

Studies were carried out initially in the laminar regime. The laminar condition of flow, insured by a low Reynolds number, was verified by the injection of dyes into the flow. The water in the systems was analyzed for oxygen concentration daily at the following four points: inlets and outlets of the pipelines, centers of the pipes and at pipe walls. Sensible differences in oxygen concentrations of up to 1.7 ppm were registered between the center of the flow and the flow at the pipe walls. Even with special care, some secondary flow was impossible to avoid during sampling. Therefore, differences in oxygen concentrations could have been even greater than 1.7 ppm in the pipes studied.

The corrosion curve for one of the tests in laminar flow, in which the velocity was 0.125 ft./sec., is plotted in Fig. 6. The size of pipes, water composition, and other variables were the same as in other studies in turbulent flow, which are also shown in the same figure. The low corrosion rate and dissolved oxygen measurements indicate the variations in availability of oxygen to the metal surface in different regimes of flow. In laminar flow oxygen reaches the pipe walls mainly by molecular diffusion. The corrosion rate was nearly constant. Fig. 6 shows that the curve varied upward slightly near the end of the study, because during this period fewer specimens remaining in the systems allowed more oxygen to be available per unit area of exposed metal. The corrosion curve in laminar flow did not follow the logarithmic law, as in turbulent flow, because the usual corrosion product layer did not form. Only a gelatinous and easily removed brown deposit was observed. This is in sharp contrast to the thick adherent corrosion product layer formed under conditions of turbulent flow.

2. Turbulent Flow

The majority of the studies were carried out under conditions of turbulent flow. Many pH levels were investigated, but only the results at pH 8.0 are reported herein, since the conditions of turbulent flow and pH 8.0 are often encountered in pipelines carrying domestic or natural waters. Conditions at other pH levels will be reported in a later paper. The results of studies at velocities of 1, 2, 3 and 4 ft./sec., which are all in the turbulent flow regime,

are shown in Fig. 6. The corrosion rates increased with increase in velocity of flow in this velocity range. It is seen in the figure that corrosion weight loss was not a linear function of time, but that the corrosion rate was considerably lessened after many days of pipe exposure to the flowing water.

As corrosion proceeded in these studies, a layer of rust was built up, which became thicker and denser with time. A thickness of approximately 0.05 inches was reached after 40 days of exposure. The mechanics of flow analyses show that a laminar sub-layer can exist in the case of turbulent flow in pipes, but the formation of a rough corrosion product layer can generally be considered as disrupting this sub-layer. Many investigators, in reporting the influence of velocity on corrosion rates, have explained their results in a very vague way by referring to a liquid film that decreases in thickness as the velocity increases. In the case of turbulent flow in pipes, it is not easy to define such a film and even more difficult to visualize it.

The presence of a corrosion product layer on the metal surface results in the existence of two interfaces. These are at the metal-corrosion product zone and at the corrosion product-water zone. The corrosion influencing reactions taking place and the chemical products in solution or in suspension at the two interfaces are quite different. Little, if any, flow of water can be expected to exist within the corrosion product layer, because the pressure gradient to induce such a flow cannot be produced. Eddies due to turbulence can affect only a very narrow zone near the surface of the layer.

Thus in the case of normally turbulent flow it is necessary to consider a well agitated core of water flowing within a corrosion product layer containing an entrapped film of water. Since there is very slight agitation, if any, within the film, nearly all transfer of matter must take place by diffusion. This situation produces concentration gradients. Thus the chemical substances in the water entrapped in the corrosion product layer and the substances in the flowing water will differ in both concentration and composition. The corrosion product layer offers resistance to the transfer of matter, and thus suspended or colloidal particles will be filtered out by adsorption in the capillary size channels. Dissolved chemicals, such as oxygen and metallic ions, may be partially consumed to form other compounds before reaching the surface of the metal or the main water channel. Stumm[23] has reported that calcium carbonate is not uniformly distributed in the corrosion product layer on iron surfaces. The solubilities of the corrosion products vary with environmental conditions such as pH, as shown by Forrest, et al.[24] It is therefore seen that the formation of a corrosion product layer, while generally reducing the rate of corrosion as it builds up to a rather impermeable protective coating on the metal, may be more or less protective according to the environmental conditions present.

Corrosion rates are plotted at velocities of 1, 2, 3 and 4 ft./sec. in Fig. 7. They were calculated for periods of 10 and 40 days after the beginning of the corrosion tests. Although there was a general increase in corrosion rate with increasing velocity, the difference between the 10-day and the 40-day corrosion rates was significant. During this time interval, the formation of a protective corrosion product layer decreased the corrosion rate by more than 50 per cent.

3. Effect of Reynolds Number

In Table I are presented the corrosion rates obtained with various Reynolds numbers. One can observe that at the same Reynolds number the

Table I

CORROSION RATES AT 40DAYS AT VARICUS REYNOLDS NUMBERS

Pipe Diameter inches	Velocity ft./sec.	Reynolds Number	Corrosion Rate mg./sq.dm./day
0.75	1	5,720	57
0.75	2	11,440	65
0.75	3	17,160	73
0.75	4	22,880	101
1.0	2	15,300	62
1.5	2	22,880	216

corrosion rates are different with different pipe diameters. From theoretical considerations it can be predicted that the Reynolds number is not a criterion of similarity in corrosion processes. The results of this experimental research show that this applies to flow in pipes.

Higher corrosion rates in larger diameter pipes can be explained by the increase in the intensity of mixing, which depends upon the mixing length, l. However, at the same velocity, the velocity gradient decreases with increasing diameter. In addition, the ratio of the volume of corrosive water to the area of pipe surface exposed, or the hydraulic radius, increases with increasing pipe diameter. Thus there is a relative increase in oxygen available in larger pipes.

It can be seen that the corrosion rates, dosages of inhibitory chemicals, etc., which have been or may be determined in model studies, will be different from those in pipes of other diameters than those used in the research studies. In general it is to be expected that increasing the pipe diameter will increase the mixing and diffusion conditions and will result in a reduction in the optimum dosage as determined with smaller laboratory pipes. The determination of the law which follows the scale of the model will require more data at different pipe diameters.

4. Corrosion Rate

Seven identical studies were made at a velocity of 2.0 ft./sec. for the purpose of evaluating the corrosion curve with accuracy. The average corrosion weight loss results from the seven tests are shown in Table II.

TABLE II

WEIGHT LOSSES IN BLACK IRON PIPES
AT VELOCITY OF TWO FT./SEC.

Time from Start of Test Days	Total Weight Loss mg./dm.^{2}
0	0
7	1900
14	3500
21	4900
28	6100
35	7000
42	7800
49	8500

Inspection and measurement of the corrosion product layer indicated that not only its thickness increased with time, but that also its physical characteristics changed as the studies progressed. A few hours after the beginning of each study a gelatinous brown layer, probably ferric hydroxide, was formed. After the first day a black layer appeared under the brown one.

During the first few days the corrosion product layer was found to be very soft and easy to remove under a water jet. The compactness increased with time, however, and after five weeks it was possible to slip the film out from the pipe all in one piece without its being broken. The layer was composed

of a thin outer surface of brown ferric hydroxide over a thicker layer of black magnetite. Changes in both thickness and physical characteristics were great at the beginning of the tests, but became less noticeable as the studies progressed.

If the term P is a coefficient which measures the ability of the corrosion product layer to allow oxygen diffusion, and if the term y is the thickness of the layer, the rate at which oxygen will diffuse, other variables remaining the same, will be:

$$\frac{do}{dt} = \frac{P}{y} \tag{38}$$

Both P and y
a decreasing rate, and the term y will increase with time at also a decreasing rate. Thus, the second term of the above equation will be a decreasing function of time. The fact that both the physical properties and the thickness of the layer vary with time, as above, suggests that the function of $\frac{P}{y}$ is one of the form e^{-kt}.

The corrosion rate, taken as proportional to the oxygen transfer rate, can then be expressed as follows:

$$h = \frac{dc}{dt} = Ce^{-kt} \tag{39}$$

and integrating between the limits of 0 and t results in:

$$c = K(1 - e^{-kt}) \tag{40}$$

where $K = \frac{C}{k}$.

Numerical values of K and k were determined in order to obtain the curve which best fits the distribution of points. The equations proved to be in good agreement with the manner in which corrosion proceeded, some of the results of which are shown in Table II.

The above equations made it possible to obtain the value of the corrosion rate at any time from the expression:

$$h = \frac{dc}{dt} = Kke^{-kt} \tag{41}$$

The initial corrosion rate would be when t = 0. It was observed in the studies, however, that generally the initial corrosion rates, such as after 2, 6, 12 and 24 hours, were slightly higher than the value obtained from the equation at time zero. The difference between values derived from the equation and experimental results at the early part of the tests are not surprising. The equation assumes the existence of a corrosion product layer whose characteristics change as corrosion proceeds. At the very beginning of the studies this layer did not exist.

Considering the equation:

$$\lim_{t \to \infty} K(1 - e^{-kt}) = K \tag{42}$$

it would appear that the corrosion losses tend to reach a maximum determined value of K. In general, this is not true because it is known that corrosion proceeds indefinitely with time. The interpretation made is that during the period in which the formula applies the reaction proceeds toward a state where the corrosion rate becomes nearly constant. Ellinger and coauthors[22] found in a ten year experiment with apparatus similar to that used in these studies that corrosion rates reached a constant value after approximately four years. Due to differences in composition of the water in Ellinger's study, the corrosion process may have been slower than in the research presented herein, and the formation of uniform corrosion product layers may have demanded longer periods of time than observed in these studies. However, they substantiate the proposal of the formation of a final equilibrium in the corrosion product layer, as indicated by the asymptotic nature of the above equation.

CONCLUSIONS

On the basis of the results obtained in the experimental studies, data in existing literature, and the approach made from theoretical fluid mechanics, it may be concluded that:

1. Velocity by itself is not a basis for comparing corrosion testing results under hydraulically different conditions. Model studies which closely simulate field conditions should be used as much as possible to avoid misleading results.

2. Assuming that an increased supply of oxygen increases the corrosion rate within the limits of oxygen concentrations found in natural waters, and that corrosion product deposits on the metal surface are not impermeable, the following deductions were made from theoretical fluid mechanics:

   a. With flat plates in laminar flow, the corrosion rate is proportional to the velocity raised to the 1.5 power.
   b. With rotating disks in an infinite fluid, the corrosion rate is proportional to the angular velocity raised to the 0.5 power in laminar flow and to the 0.8 power in turbulent flow.
   c. In smooth pipes in turbulent flow and the same Reynolds number, the corrosion rate will be proportional to the first power of the velocity. With the same velocity the corrosion rate will be inversely proportional to the diameter raised to the 0.125 power, and with the same diameter the corrosion rate will be proportional to the velocity raised to the 0.875 power.
   d. In rough pipes in laminar flow, corrosion will increase with increased turbulence, which increases with velocity.
   e. The Reynolds number is not a criterion of similarity in corrosion processes in the case of turbulent flow in rough pipes.

3. The nature of the corrosive media may produce different effects when the velocity is changed. This behavior may be attributed to changes in the solubilities of metallic salts at different pH levels. At pH 8.0, corrosion increased with increased velocities.

4. When a corrosion product layer is formed, the reactions leading to continued corrosion take place inside the layer, where the conditions may be very

different from those encountered in the main flow. No flow of importance can exist in the corrosion product layer, and the transfer of chemicals such as oxygen is accomplished mainly by diffusion. Mechanical effects of the water flow on anodes and cathodes are thus of little importance.

5. It may be assumed that at least four distinct phases exist in the corrosion process:

a. Phase One - Anodic and cathodic areas will form. During this period, there is not yet a protective corrosion product layer present, and velocity increases will generally increase the corrosion rate.
b. Phase Two - The anodic and cathodic areas increase in size and in number, and protective layers are formed. Corrosion in this period will follow a complex law, which will probably vary with different pH and velocity values.
c. Phase Three - The corrosion product layer is uniformly extended over the metal surface. The corrosion process can be expressed mathematically by an exponential relation of the form:

$$c = K(1 - e^{-kt}) \tag{40}$$

d. Phase Four - A state is reached where corrosion proceeds almost uniformly at a very low rate as long as there are no changes in any of the environmental conditions.

The length of each of the above phases depends upon the chemical quality of the water and also upon the velocity and other hydraulic properties.

6. Velocity and size of the pipe affects the corrosion rate by changing the rate of supply of oxygen and other chemicals in the water. It is to be expected that the dosage of inhibitory chemicals for the reduction of corrosion can be reduced in a certain proportion as the size of the pipe increases. Very high velocities can modify or prevent the formation of protective layers of corrosion products by their mechanical effects.

ACKNOWLEDGMENT

The research reported in this paper was supported by the John B. Pierce Foundation, National Science Foundation, Engineering Foundation and American Iron and Steel Institute.

REFERENCES

1. Camp, T. R., "Corrosion and Corrosion Research," Proc. Am. Soc. of Civil Eng., Vol. 81 (May 1955).
2. Cox, G. L. and Roetheli, B. E., "Effect of Oxygen Concentration on Corrosion Rates of Steel and Composition of Corrosion Products Formed in Oxygenated Water," Ind. and Eng. Chem., 23, 1012 (1931).
3. Uhlig, H. H., Triadis, D. N. and Stern, M., "Effect of Oxygen, Chlorides and Calcium Ion on Corrosion Inhibition of Iron by Polyphosphates," J. Electrochem. Soc., 102, 59 (1955).

4. Friend, J. A. N., "A Colloid Theory of the Corrosion and Passivity of Iron, and of the Oxidation of Ferrous Salts," J. Chem. Soc., 119, 932 (1921).

5. Friend, J. A. N., "A New Theory of the Corrosion of Iron," Trans. Am. Electrochem. Soc., 40, 63 (1921).

6. Friend, J A. N. and Dennett, J. H., "The Rate of Solution of Iron in Dilute Sulfuric Acid both when Stationary and Under Rotation," J. Chem. Soc., 121, 41 (1922).

7. Speller, F. N. and Kendall, V. V., "A New Method of Measuring Corrosion in Water," Ind. and Eng. Chem., 15, 134 (1923).

8. Whitman, W. G., "Corrosion of Iron in Water," Chem. Reviews, 2, 419 (1925).

9. Wilson, R. E., "The Mechanism of Corrosion of Iron and Steel in Natural Waters and the Calculation of Specific Rates of Corrosion," Ind. and Eng. Chem., 15, 127 (1923).

10. Evans, U. R , "Oxygen Distribution as a Factor in the Corrosion of Metals," Ind. and Eng. Chem., 17, 363 (1925).

11. Roetheli, B. E. and Brown, R. H., "Corrosion Rates of Steel and Composition of Corrosion Products Formed in Oxygenated Water as Affected by Velocity," Ind. and Eng. Chem., 23, 1010 (1931).

12. Frese, F. G., "Effect of Oxygen on Corrosion of Steels," Ind. and Eng. Chem., 30, 83 (1938).

13. Speller, F. N., "Principles of Corrosion and Its Prevention," J. New Eng. Water Works Assoc., 52, 231 (1938).

14. Wormwell, F. J., "The Influence of Movement on the Corrosion of Metals in Salt Solutions and Natural Waters," Iron and Steel Inst., 154, 219 (1946).

15. Hatch, G. B. and Rice, O., "Corrosion Control with Threshold Treatment," Ind. and Eng. Chem., 37, 752 (1945).

16. Copson, H. R., "Effect of Velocity on Corrosion by Water," Ind. and Eng. Chem., 44, 1745 (1952).

17. Streicher, L., "Effects of Water Quality on Various Metals," Jour. Am. Water Works Assoc., 48, 219 (1956).

18. Uhlig, H. H., The Corrosion Handbook, John Wiley and Sons, New York, (1948).

19. Schlichting, H., The Boundary Layer Theory, McGraw-Hill, New York (1955).

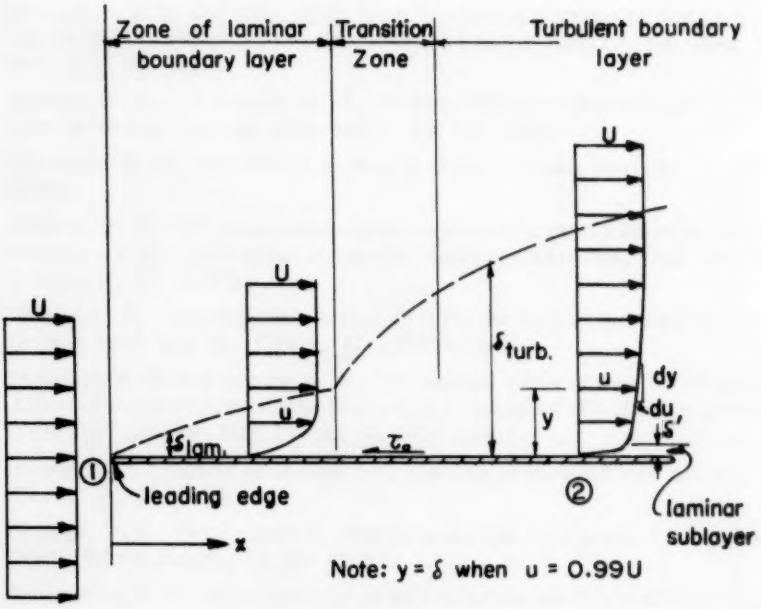
20. Rouse, H., Engineering Hydraulics, John Wiley and Sons, New York (1950).

21. Nikuradse, J., "Stromungsgesetze in rauhen Rohren," V.D.I. Forschungsheft 361, (1933).

22. Ellinger, G. A., Waldron, L. J. and Marsolf, J. B., Proceedings Am. Soc. Testing Materials, 48 618 (1948).

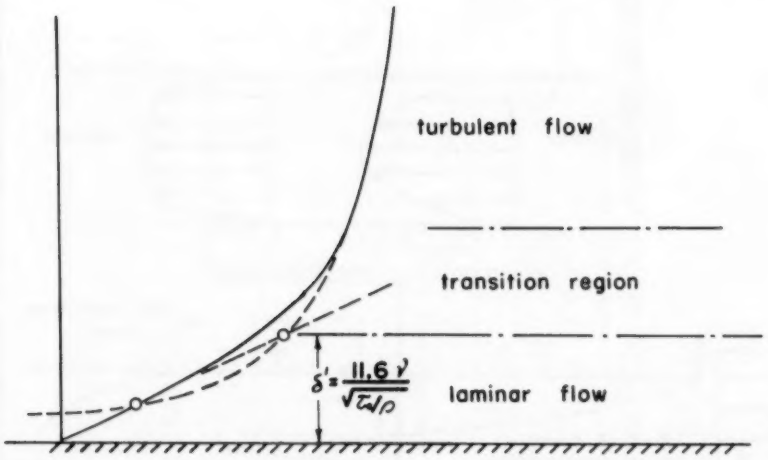
23. Stumm, W., "Calcium Carbonate Deposition on Iron Surfaces," Jour. Am. Water Works Assoc., 48, 300 (1956).

24. Forrest, H. O., Roetheli, B. E., and Brown, R. H., "Products of Corrosion of Steel," Ind. and Eng. Chem., 23, 650 (1931).

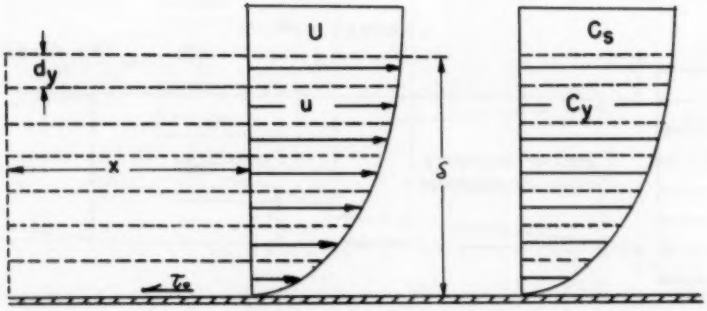


DEFINITION SKETCH FOR DEVELOPING BOUNDARY LAYER

FIG. 1



VELOCITY DISTRIBUTION IN TURBULENT FLOW



OXYGEN DEPLETED LAYER

FIG 2 FLOW OVER A FLAT PLATE

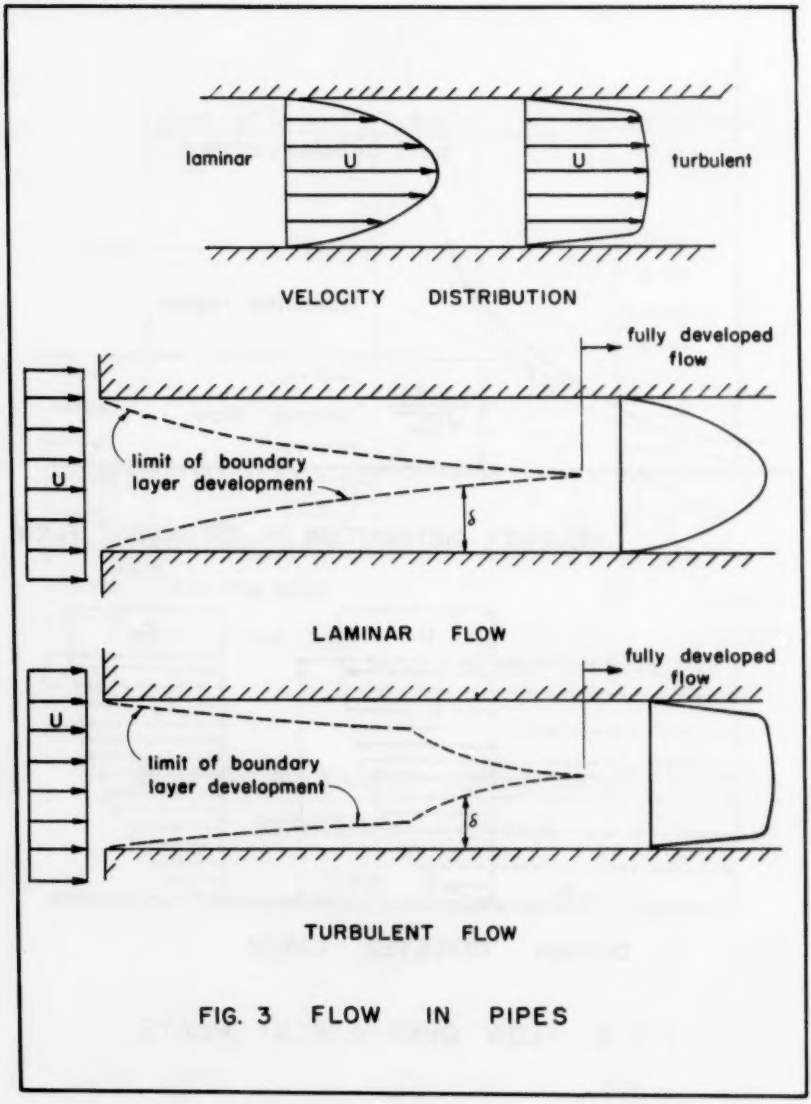


FIG. 3 FLOW IN PIPES

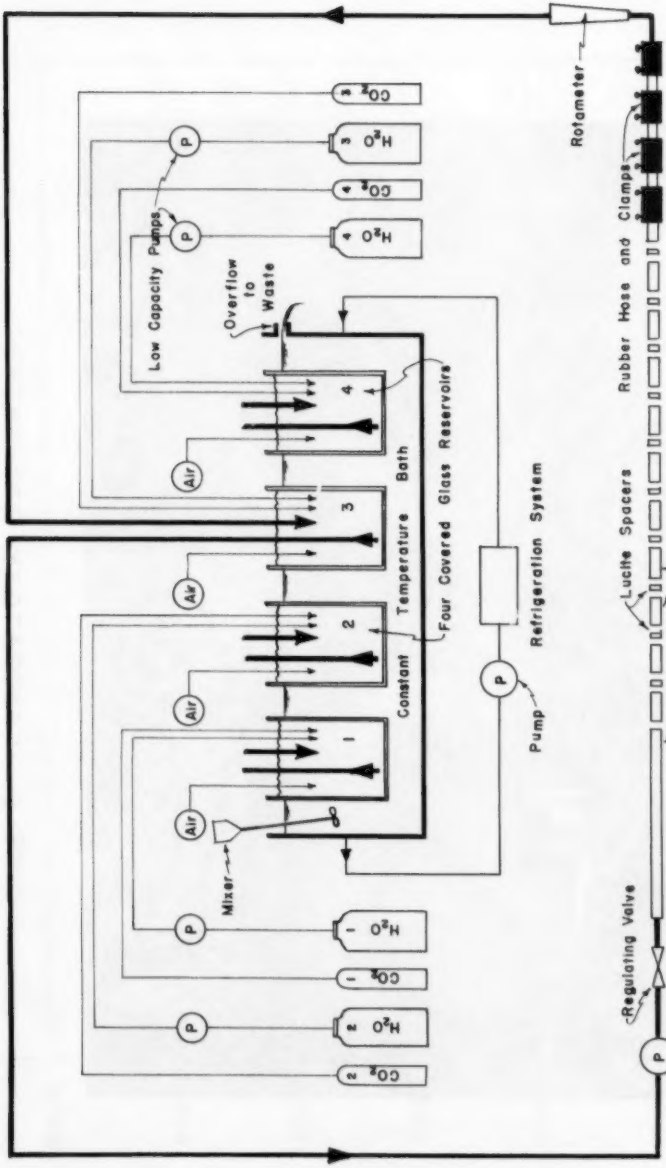


FIGURE 4
FLOW DIAGRAM

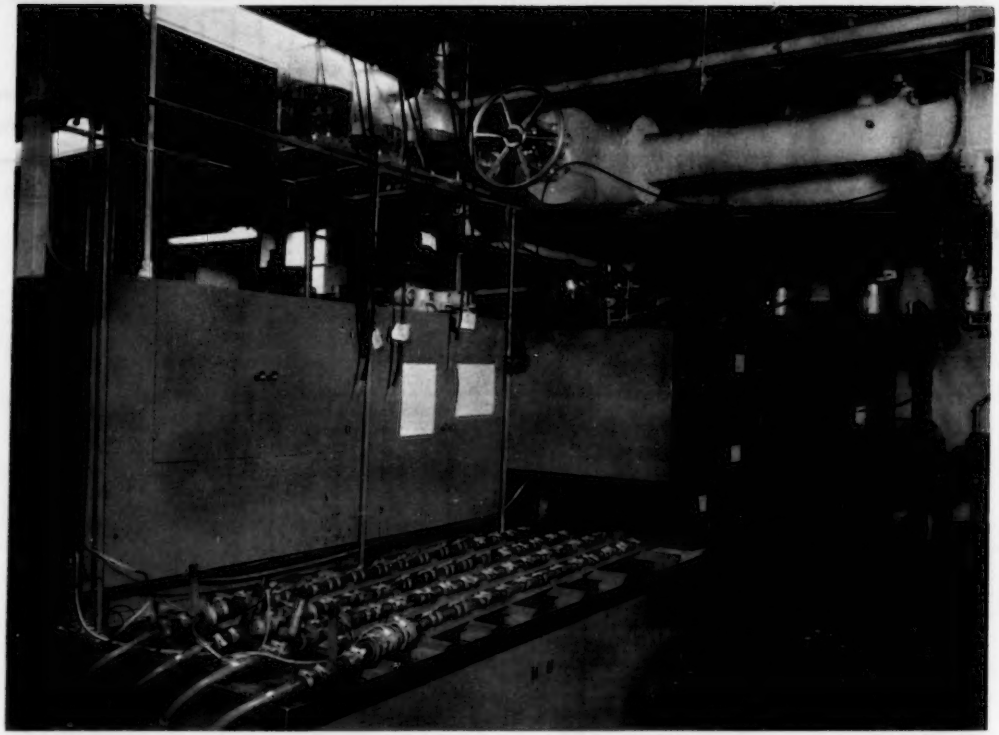


FIGURE 5 LABORATORY APPARATUS

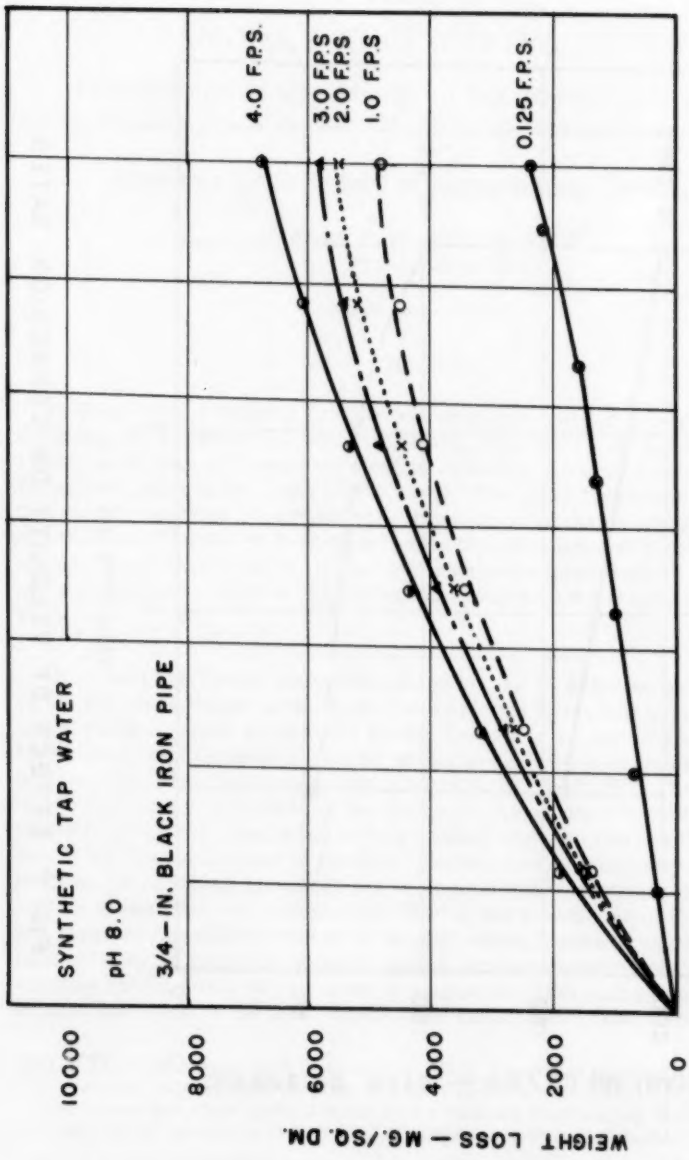


FIG. 6 EFFECT OF VELOCITY ON CORROSION OF IRON PIPES

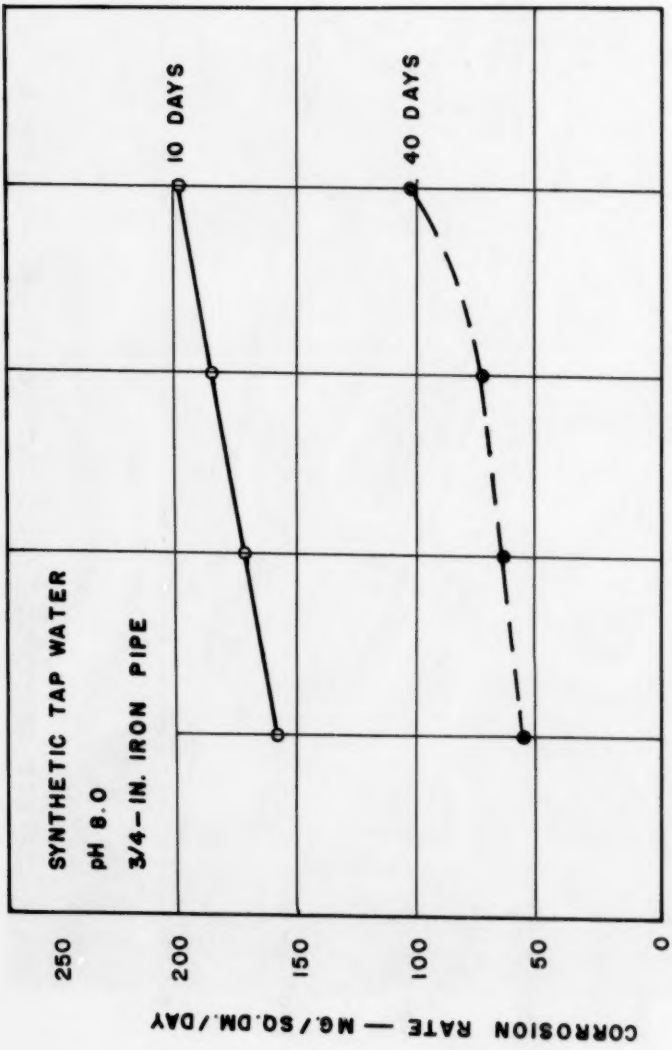


FIG. 7 EFFECT OF VELOCITY ON CORROSION RATES

Paper 1703 July, 1958 SA 4

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

FORCED CIRCULATION OF LARGE BODIES OF WATER[1]

Thomas M. Riddick, M. ASCE[2]
(Proc. Paper 1703)

ABSTRACT

In June 1956, a floating aerator was designed and put in service at the Ossining, N. Y. Municipal water collecting reservoir. An eight HP blower connected to this unit was sufficient to circulate the entire volume of water — 80 million gallons, at a rate of 153 MGD. The most important results were — completely breaking reservoir stratification; retardation and substantial elimination of winter-freeze-over: reduction in color and carbon dioxide; oxidation and precipitation of iron and manganese; increase in dissolved oxygen content; and a shift of the biological environment towards the aerobic side.

It is well and widely known that large bodies of water stratify, and in this paper the term "large bodies" will be employed to include natural fresh water lakes and man-made reservoirs having a depth in excess of about twenty-five feet. This stratification of course is due to variations of water density with temperature, density being a maximum of 4.0°C or 39.2°F. Reservoirs within the United States, and those of the Northern Hemisphere between the approximate longitudinal boundaries of this country undergo two overturns yearly, one in the spring and one in the fall. During, and immediately following, these periods the physical, chemical and biological characteristics of water from bottom to top approach uniformity. During the remainder of the year there are three rather distinct zones of stratification; a zone of daily to weekly overturn existing in the upper third; a zone of transition known as the thermocline, existing below; and a bottom zone of stagnation often occupying somewhat greater than one third of the total depth of the reservoir. Thus a natural lake in

Note: Discussion open until December 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1703 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 4, July, 1958.

1 - A paper presented at the October 1957 Meeting of the ASCE in New York City

2 - Cons. Engineer and Chemist, New York, N. Y.

New York State about thirty feet in depth will have in mid June a temperature of approximately 65°F in the zone of circulation, 65° to 45°F from top to bottom of the zone of transition and 45°F throughout the zone of stagnation.

One may thank Whipple, Hazen, Ward, Smith, Birge, Welch and many others for their definitive studies in limnology and biology which today give ample working tools for predicting and evaluating the normal seasonal changes in stored waters.

Milo Churchill of TVA has within the past year given us our first published comprehensive picture of the modifications imposed when reservoirs are employed for hydroelectric power development. When reservoir detention is reduced for 3 — 6 months to 5 — 15 days, the laws of static reservoirs are rigorously upset and each reservoir becomes a separate and rather distinct problem.

Aerator Installation

In late July of 1956 a unique type of floating aerator designed by the writer was installed in the Ossining, N.Y. Municipal water collecting reservoir. Figures 1 and 2 show a closeup and a distance view of the aerator in service. The reservoir has a length of 2200 feet, an average width of 400 feet and a depth at the lower end approximating 30 feet. A plan and section of the reservoir, its capacity curve and the location of the sampling points are shown on Figure 3. There was no appreciable inflow or outflow for the first month of operation. The aerator (in reality functioning as an air lift pump operating at a very low head) produced a continuous upwelling of low lying waters at a rate of 107,000 gallons per minute, with only 8 HP of power input. Continuous circulation was effected and maintained and all thermal stratification was broken. Since storage in the reservoir at the time approximated 80 million gallons, the rate of recirculation is equivalent to two overturns per day.

Figure 4 shows details of construction of the aerator. This was 40 feet long and air releasers were spaced at one foot centers. Depth of submergence approximated eight feet. Air was supplied from a 160 cfm compressor located in the filter building and fed to the aerator through two 400 ft. lengths of plastic hose, which were wired together and floated on the water surface. The aerator was located mid width of the reservoir, 450 feet from the deep end and 1750 from the shallow end. The throw of water was a full 1750 feet to the shallow end as was demonstrated by bottle floats submerged to cork level. Needless to state the fundamental and established laws of limnology do not now apply to this reservoir, except in a limited sense. It will require many years of study of many such reservoirs to fully evaluate this decided alteration of nature's way of doing business.

Effects of Aeration

Fundamentally, the changes brought about by continuous recirculation are as follows: (a great many of these changes were anticipated and follow simple laws of physical, organic and inorganic chemistry).

1) - Thermal stratification is completely broken and temperatures from top to bottom are essentially uniform at any given time.

Figure 5 shows water temperature at sampling point No. 3 before the aerator was placed in service, and for seven days thereafter. Stratification to a depth of 20 ft. was completely broken in three days, whereas it required seven days to put the bottom waters in circulation.

This sampling point was 350 feet north of the aerator.

2) - There is a daily temperature change throughout the year varying from a few tenths to 1 - 2 degrees Fahrenheit. The change is caused by hourly air temperatures, (which generally vary from minimum during the early hours of the morning to maximum during early hours of the afternoon), number of hours of sunshine in one day and the intensity of the sun's rays reaching the water surface. The two latter factors have greater effect; thus, a dark and cloudy day produces little change, with maximum change effected during cloudless days of very low humidity. Those seriously interested in outdoor photography can attest to the fact that some haze is nearly always present, and in Metropolitan New York a good photograph of an object a mile distant cannot be taken more than one or two days per month. According to Hausmann and Slack[3] "in full sunlight, the energy received above the earth's atmosphere is about 2 calories per minute on a surface 1 cm square held perpendicular to the sun's rays; about one-third of this amount is absorbed in passage through the atmosphere". It is worthy of note that one hour of sunlight at the above intensity, if fully absorbed at the water surface of the Ossining reservoir, (with no radiation or other heat losses) would raise the temperature of the entire 80 million gallon volume almost 1/2 degree Fahrenheit, representing an equivalent of 90,500 horsepower. The large amount of energy potentially available from the sun is a challenge to scientists, and unless atomic energy furnishes the need, some device for harnessing sunlight seems to be a most promising means of furnishing power after exhaustion of the definitely limited and fast dwindling coal, oil, gas and shale resources.

3) - Average weekly water temperatures in general follow and naturally lag behind the average weekly air temperatures. The effect of average air temperatures can be overridden by plus or minus one or two degrees by the effect of sunshine.

4) - The lag in water temperature is probably affected to a slight degree by the heat transferred by the muck and sand constituting the reservoir bottom. Although this is of no moment during the spring, summer and fall, it may play an appreciable role in the winter months in retarding freezing of the water surface. Figure 6 shows temperature changes at sampling points 2, 3 and 4. The lag of the bottom water is evident.

5) - Turbidity was lowered slightly. The two feeder brooks entering the head end of the reservoir carry only a slight silt load, and turbidity has always been low — in the range of a few part per million. The further lowering due to recirculation is judged to be occasioned by more rapid oxidation and dropout of dead algae and decomposing vegetation categorized in microscopical analysis simply as amorphous matter as well as less algae.

6) - True color initially averaged in the range of 55-60 ppm. It was found by Whipple, Hazen, Stearns and others, about the turn of the century, that sunlight tends to bleach color, but that this effect takes place in surface waters only and is confined to a depth of only a few inches. The continuous pumping of low lying waters to the surface naturally

[3] Hausman-Slack Physics — D. Van Nostrand Co., Inc.

permits this action to proceed to a much greater degree than heretofore, with the result that color is reduced to 20 — 35 ppm. The variation is undoubtedly a function of hours of sunlight, rainfall and water consumption, stage of the reservoir, swamp drainage and related factors.

7) - The carbon dioxide content of the reservoir waters was markedly changed. In a stratified state the surface waters normally contained 1 — 2 ppm, increasing to 10 — 20 ppm in the zone of stagnation. Continuous circulation and the continuous upwelling of bottom waters to the surface now permits carbon dioxide to escape to the atmosphere as fast as it is formed. This naturally results in a relatively constant CO_2 content throughout the reservoir of 1 — 2 ppm. Since pH is a function of carbon dioxide and alkalinity, the lowering of CO_2 through aeration increased pH from a previous value of approximately 6.0 in the zone of stagnation, to a uniform value of about 6.9 throughout the reservoir.

8) - Prior to continuous circulation, Iron approximated 0.05 — 0.2 ppm in the zone of circulation, increasing to values of 2 — 4 ppm in the bottom waters of the zone of stagnation. In normal reservoirs, Iron is oxidized in the zone of circulation to ferric hydrate, due to the high oxygen potential. This action is aided by the relatively high pH and low CO_2 which also prevail. Upon precipitation through the thermocline to the zone of stagnation, Iron is then reduced to ferrous bicarbonate, due to low dissolved oxygen, low pH and high CO_2. The high CO_2 is of course due to decomposition of organic matter. Recirculation, with its attendant aeration, does not permit iron to be reduced to a soluble state, and to the contrary the high oxygen potentials throughout the reservoir result in precipitation. The soluble iron originally present in the Ossining reservoir was definitely oxidized, and precipitated probably in the far ends of the reservoir where velocities were minimum.

The concentration of manganese in the Ossining water was initially too low to furnish conclusive data as to its ultimate disposition. Whereas Iron in solution is readily oxidized to an insoluble state by a spray aerator, manganese is oxidized so slowly that spray aeration is entirely ineffective. It is probable however that manganese would be oxidized and would precipitate in a recirculated reservoir during the two to eight months of storage which often obtains.

9) - Dissolved oxygen was formerly 95 to 100 percent saturated in the zone of circulation with bottom waters approximately depleted of oxygen, in the range of 0 — 0.5 ppm. This is a normal condition for most reservoirs in the United States. Since depth samples had been collected at 5 ft. intervals from top to bottom at seven sampling points prior to placing the aerator in service, the total poundage of DP of the reservoir was well established. It was hoped that D.O. determinations after the aerator was put in service might indicate or establish the rate of reaeration, but this proved entirely impossible. After the stagnant bottom waters (depleted of dissolved oxygen) were removed from the muck-water interface and replaced with waters 80% or more saturated, a relatively high biological oxygen demand of the muck was clearly evident.

Undoubtedly the faculative aerobes and anaerobes constituting the predominant bacterial types in the upper layers of muck, and at the

muck-water interface, died off and were replaced by aerobic types of bacteria and other microorganisms. It must be borne in mind that the reservoir had been in service 68 years and that the entire bottom was covered with muck representing the slight silt load of the inlet feeder brooks; plus decayed and decaying remains of plant and algal growths; plus the organic matter constituting color, part of which eventually precipitates. The constituents constitute an organic muck loading hard to define and difficult to evaluate. The upper or shallow end of the reservoir was entirely in the zones of circulation and transition, hence water at the muck interface contained from 40 to 100% saturation of dissolved oxygen.

Muck in the deep sections at the lower third end of the reservoir, however, were subject to overlying waters varying from no oxygen to only a few tenths or a few parts per million. Under such circumstances it is doubtful that even relatively complete oxidation could be effected. When viewed with hindsight therefore, it is not surprising that after circulation was created, dissolved oxygen in the entire reservoir dropped to a level of 80-85% saturation, or that three or four months are required to raise this level to the 95-100% level. Even after about one year of operation it is probable that there is some oxygen demand, for undoubtedly organic solids at the muck-water interface are first oxidized, then oxidation is effected at succeedingly greater depths — eventually perhaps to the earth layer constituting the original bottom.

Figure 7 shows the rapid rise in dissolved oxygen at twenty foot depths at sampling points 2, 3 and 4. The lag at the 25 ft. depth is quite evident. Figure 8 shows the rise in dissolved oxygen at sampling point 3. This was effected in 2 days for depths up to 20, whereas 7 days were required for aerating the 25 ft. depth. It must be borne in mind that circulation of 330,000 tons of water was produced by an 8 HP input.

Color furnishes an organic load, some of which will eventually precipitate due to oxidation and to bacterial activity. This is particularly evident when working with waters from the Dismal Swamp in North Carolina and Virginia where color ranges from 1000 to 2000 ppm. The slow decomposition of submerged organics is also attested to by the work of Daknowki-Stokes on muskeg and peat swamps. A microscopical examination by the author of a one foot square — one foot deep section cut from a muskeg in Alaska revealed copious organics, still green at a depth of one foot below the surface.

Continuous overturn of reservoir or lake waters therefore does create an entirely different environment at the bottom, and this environment is characterized by very high rather than very low oxygen potentials. Nature seems to prefer to effect organic decomposition by aerobic bio-chemical means with the ultimate formation principally of carbon dioxide, water, nitrates and mineral compounds. If oxygen is completely absent, then decomposition can also proceed by anaerobic means (as in sludge digesters), the principal gaseous end products being methane and hydrogen.

Between these two extremes lies an intermediate biological zone — one characterized by slight aerobism and slight anaerobism. The muck at the water-muck interface in the deep end of most reservoirs

presents such an ecological environment. Oxygen generally varies from a few tenths of a ppm to complete depletion, and under these circumstances facultative aerobes and anaerobes are able to break down the sulphate radical to obtain oxygen, thereby forming hydrogen sulphide.

This condition occurs at a few water storage reservoirs where the organic loading is too high to be satisfied by the limited oxygen residuals inherent to the bottom waters of the zone of stagnation. It is logical therefore that forced circulation, which results in complete aerobism at the muck-water interface, represents a favorable change in the biological environment.

There are still a great many municipalities and industries employing chlorinated raw reservoir water. New York City is a striking example. The useful capacity of such waters is that contained in the zones of circulation and transition only. Waters from the zone of stagnation cannot be employed because of high color, high iron and sometimes high manganese. It would seem that forced circulation would increase the potential yield of such reservoirs by about one-third, since it also makes available the water from the zone of stagnation.

A man-made change in biological environment must be approached with utmost care and consideration, for it is a fundamental alteration of nature. In contemplating the ultimate effects of recirculation of reservoir waters upon the biological balance, it was believed that for any given reservoir, the incidence of algae would be decreased through a lowering of carbon dioxide, which with sunlight and through photosynthesis enables plant and algal growths. The opposite tendency, that is, stimulated algal growth, would however also be encouraged by some lowering of turbidity, which permits sunlight to penetrate greater water depths. Mr. Charles Merritt, who has been superintendent of filtration at Ossining for thirty years, concludes that algae, broadly speaking, have been reduced by 50 - 75%. The same types are still present, but total concentration has been lowered. Copper sulphate has been applied only once since reservoir recirculation was commenced, and this was due to the introduction of several million gallons of Croton Aqueduct water (to the Ossining Reservoir) containing 5,000 to 15,000 standard areal units of Anabaena. This was at a time when Croton Lake was drawn down to levels of the stagnation zone. The species of algae and Protozoa normal at Ossining include Asterionella, Navicula, Tabellaria, Volvox, Pediastrum, Clathrocystis, Coelosphaerium and very occasionally the very objectionable Dinobryon and Synura.

Aside from a decrease in total algal counts, and a decrease in the concentration of amorphous matter, there were two other significant changes. Crustacea, including Cyclops, Bosmina and Diaptomus more than doubled.

About two months after the aerator was installed, the concentration of Diaptomus was 5 to 10 times normal, representing a concentration of 5 to 10 organisms per liter. This decreased over the next few months however and now the incidence of Rotifera seems to have leveled off at 2 or 3 times the previous normal in the stratified reservoir. It is probable that the increase was due largely to an increased source

of food supply, for now the entire reservoir bottom is ecologically habitable. A great deal more is known of Daphnia than other Crustacea. They are thought to utilize bacteria as a source of food supply. The principal Daphnia agent in Metropolitan New York, who supplies many aquarium supply shops, advises me that his best pond receives the discharge of large volumes of laundry wastes; also that Daphnia growth can be greatly stimulated in any pond by tying a rock to a dead rabbit or other sizable animal, and submerging it in the pond. It is likely that the increase in Daphnia is through the increase of bacteria occasioned by the presence of animal protein.

At the present time, Mr. Merritt advises that the Ossining Reservoir contains many times the concentration of Smallmouth Black Bass fingerlings ever heretofore noted. This may be due to an enlarged ecological environment available to the fingerlings per se; or it may be due to stimulated bacterial growth in the bottom muck occasioned by the high oxygen content, which in turn increases the food supply and incidence of Crustacea, which in turn increases the food supply for the Bass.

10) - Perhaps the greatest changes effected by continuous overturn are related to freezing. In stratified reservoirs during the fall months, as average day and night temperatures progressively decrease, the water at the reservoir surface becomes cooled, and due to increased density settles to the bottom. This action continues until the fall overturn is effected, at which time temperature from bottom to top closely approximates 4.0°C (39.2°F). Since further cooling decreases the density, no further water movement due to thermal changes is effected and the reservoir freezes over in a thin sheet. Further freezing of water below with the formation of thicker ice is due to heat losses through the formed ice per se, and the thicker the ice, the slower is the rate of heat transfer. Thus ice formation per se acts as an insulating barrier (which is more effective than an equivalent thickness of water) to prevent further freezing.

Most reservoirs in the Ossining area during winters of moderate or average severity freeze to a depth of about 12 inches, with mild winters producing about 8 inches and very severe winters about 16.

In the winter of 1956-57, which was about average, or slightly milder than average, reservoirs in the Ossining area froze over during the latter part of November and by mid-January had ice thicknesses of 8 to 12 inches. The Ossining Reservoir remained open through November and mid-December, when ice started to form at the shallow (brook inlet) end. The last week of December and the first week or two of January where characterized by a very cold spell, with air temperatures dropping down nightly in the range of 0 to minus 10°F. Surface ice at the shallow end then progressively increased in area until about one-third of the reservoir (the shallow section) was ice covered. This was judged to be a thickness of about six inches. Two-thirds of the area (at the deeper end) were completely open however, with little or no ice at the shore line. The following record of water temperatures, collected at about 9:00 a.m. are of interest:

December	1 – 2	43°F
"	3 – 5	42
"	6 – 8	41
"	9 – 10	40
"	11	39
"	12	38
"	13	39
"	14 – 15	38
"	16 – 17	37
"	18 – 27	36
"	28	35.5
"	29	34.5
"	30	34.0
"	31	33.5

The reservoir then maintained a fairly uniform temperature from 32.5 to 34.0°F from January 1 through January 17. From January 15 to 17 air temperatures dropped nightly to 6-9°F below zero with highs daily from plus 9 to plus 16°F. On the night of the 17th the air hose connections to the aerator blew off and by morning the entire reservoir was frozen over. Ice thickness then built up rapidly and within a few days was too thick to cut-out to reach the aerator. Needless to say the aerator was thereafter inoperative until the spring thaw permitted reconnecting the hose lines on March 12. If the air lines had not blown off it is possible that the aerator would have kept the lower half end of the reservoir free of ice. It is also possible that the water could have been supercooled a degree or so, and the entire water mass turned to slush or frazil ice. If this had occurred it might have created some acute problems in withdrawal by the 16 inch intake line.

One can only speculate as to the causes of the reservoir remaining free of ice cover for about two-thirds of its area. The answer would seem to lie in the phenomenon of daily absorption (and subsequent release) of tremendous amounts of solar energy. On cloudless days of bright sunshine, low humidity and low haze, this can amount to as much as 2.5°F, either winter or summer. On cloudy days there are only a few tenths °F of diurnal change. On days of moderate sunshine, such as occurred in early October, daily fluctuations were in the range of 1 to 1.5°F.

It is of interest that the bottom waters of a stratified reservoir drop no lower than 39.2°F, whereas in a recirculated reservoir they can drop 7°F below this value. During the last war, in the Public Health V-7 D. Clinics, a great many persons were successfully freed from syphillis by raising body temperature from 98.6°F to 105°F. A change in temperature of 7°F held constant for 5 to 7 days is sufficient to kill the spirochetes of syphillis, which are only accustomed to the constant temperature of their host. Aquatic microorganisms in this area however must have a normal survival range of 39.2°F in the winter and 80°F in summer. This range of about 40°F is probably so broad that the additional drop of 7° occasions no biological ill effects. If the Ossining Black Bass had chilly nights and days during the past winter, it was not sufficient to discourage spawning the following fall.

There is no doubt that stratification can be completely broken and continuous recirculation effected in reservoirs of 100 MG capacity with an HP input of only 8. And with a proportionately greater amount of horsepower, this effect can undoubtedly be achieved in reservoirs of one billion and perhaps many billions of gallons.

Figure 9 shows velocities at the surface, and at depths of one and two feet. These measurements were made 10 ft. off the aerator. It will be noted that the major portion of upwelling water is found in the top foot of depth, and that the rate of pumping was 107,000 gallons per minute.

The fundamental concepts of circulating large volumes of water with relatively small amounts of air to effect oxygen uptake and biochemical oxidation predate the Ossining installation by several years. I have maintained a 40 gallon fresh water aquarium in my home for about eight years, with tropical fish populations varying from 40 to 125. At the latter concentration, continuous aeration is absolutely mandatory to supply the oxygen demand. These studies emphasized the tremendous uptake of oxygen at the water surface, which is enabled by a small continuous upwelling of bottom waters appreciably deficient in dissolved oxygen.

Other Aerator Installations by the Author

The activated sludge process employs tremendous amounts of air and violent agitation. Since the ratio of surface area at the air-water interface at the liquid level, to the surface area of the rising stream of bubbles is relatively low, oxygen transfer is principally from the bubble surface to the surrounding water. This of course necessitates air bubbles of small size as the area of spheres increases as the square of the diameter.

If air from submerged diffusers is employed as a pumping device however, bubble size is unimportant but it is necessary that relatively large air-liquid level areas be available to receive the upwelling liquid and expose it to atmospheric oxygen.

This principle, though heretofore untried, was incorporated in the design of the Hilton-Davis Waste Treatment Plant in Cincinnati. Here the extremely corrosive nature of the waste mitigated against the extensive employment of concrete basins, and 7 to 10 days of detention were required to take stoichiometric advantage of the intermittent discharge of acids and alkalies. Also laboratory investigation indicated that reductions of BOD and chemical oxygen demand were appreciable, in the range of 60 to 80% upon 5 to 7 days of aeration, but that little oxidation was effected in detention periods normal to the activated sludge process. Three lagoons were therefore designed, one primary for seven days detention at 1.0 MGD and two secondary units at 3 days detention each. Depth was intentionally limited to about 8 feet and aerators were designed for a 100 foot throw of liquid. Power to accomplish this was rather arbitrarily set at 50 HP. Up to this time a 100 ft. throw would have far exceeded that of any known installation.

The lagoons, pumping station, pipe lines and other accessories were completed in late July 1953 and the plant had to be immediately placed in service to enable a rating by Cincinnati sewer authorities, who make a yearly charge to industry on a basis of suspended solids and BOD of the effluent. The concrete columns supporting the aerator were installed but time did not permit

installation of the aerator or blowers. A new contract was therefore negotiated with the contractor to complete the aerator installation the following summer — under operating conditions. The ensuing year of operation showed such good effects to be achieved by prolonged lagooning, 95% reduction in suspended solids and reduction in BOD better than 50%, that decision was made by management to postpone the aerator installation until really needed. It is believed that the relatively large area of these lagoons, and relatively shallow depth, is responsible for their excellent operating results.

Figure 10 shows the Hilton Davis Waste Treatment Plant at Cincinnati. This provides 10 days detention in relatively shallow lagoons. The H seen in the large lagoon is not their trademark, but concrete piers installed to support the aerator lines.

Immediately following the Cincinnati installation there was need of additional aeration at the Ossining Waterworks to remove nitrogen chloride and other dissolved gases from the raw water following the application of 8 to 12 ppm of chlorine for taste and odor control. The spray aerators then installed discharged to a basin 100 feet long, 50 feet wide and 5 feet deep. To augment this aeration, a 100 foot long line of 4" diameter carborundum tubes was submerged 4 feet along one wall. With HP input of 8, a rolling current was established with surface water traversing the 50 foot throw in 50 seconds. Due to the relatively large surface area of 5000 square feet, and an average velocity of 1.0 foot per second overall aeration was judged far superior to that enabled by the equivalent of about 12 HP of input to the aerators. If a throw of 50 feet could be readily effected, then the Hilton-Davis aerators were most conservatively designed. It remained for the Ossining Reservoir installation to prove that instead of 100 feet the ultimate throw or drift of water could be as much as 1700 feet.

At the present time, construction is substantially complete for a waste treatment plant for Eli Lilly and Company in Lafayette, Indiana, designed by the author. The wastes are principally organic chemicals, with acids sufficient to lower pH to the range of 0.2 to 2. After a unique type of chemical biological treatment, the raw sludge will approximate 20,000 gallons per day with a BOD ranging in the thousands of parts per million. Four aerated lagoons are being employed and the author trusts that operating results will not indicate that a boy was sent to do a man's job.

Applications

The overall aeration process — for floating or stationary types — is being patented. It is believed that this principle has application for the following:

1) - In fresh water reservoirs, to lower the concentration of iron, manganese, color, and algae; and to prevent anaerobic conditions in the zone of stagnation.
2) - To supply oxygen to trade waste lagoons and to effect biochemical stabilization.
3) - To apply lime, copper sulphate, activated carbon or other desired chemicals to fresh water reservoirs, and to apply bacterial nutrients to trade waste lagoons. At Ossining, the prime purpose of the aerator was to spray lime on the water surface and convert this to calcium bicarbonate through absorption of atmospheric carbon dioxide — thus increasing "natural" alkalinity for coagulation and for corrosion control.

If ice locked harbors are underlain with fresh or sea water at a temperature one or two degrees below the freezing point, it seems possible that ice formation could be prevented in shipping lanes by appropriate aeration devices to cause an upwelling of warmer waters below. Although horsepower input might be in the range of 200 to 1000, it also seems possible to create circulation and induce appreciable aeration in our large hydro electric reservoirs where average flows are in the range of 1,000 to 10,000 cubic feet per second.

ACKNOWLEDGMENT

Special credit should go to Mr. Charles Merritt and to Mr. Victor Cordy of the Ossining Waterworks for collecting much of the data on which this paper is based.

"Addendum - At the time of publication of this paper the Ossining Reservoir has undergone its first year of continuous circulation. The absorption of solar energy substantially prevented freezing during the winter of 1957-58. Ice formed only in the shallow end, representing say one-quarter to one-third of the total surface area".

FIG. 1

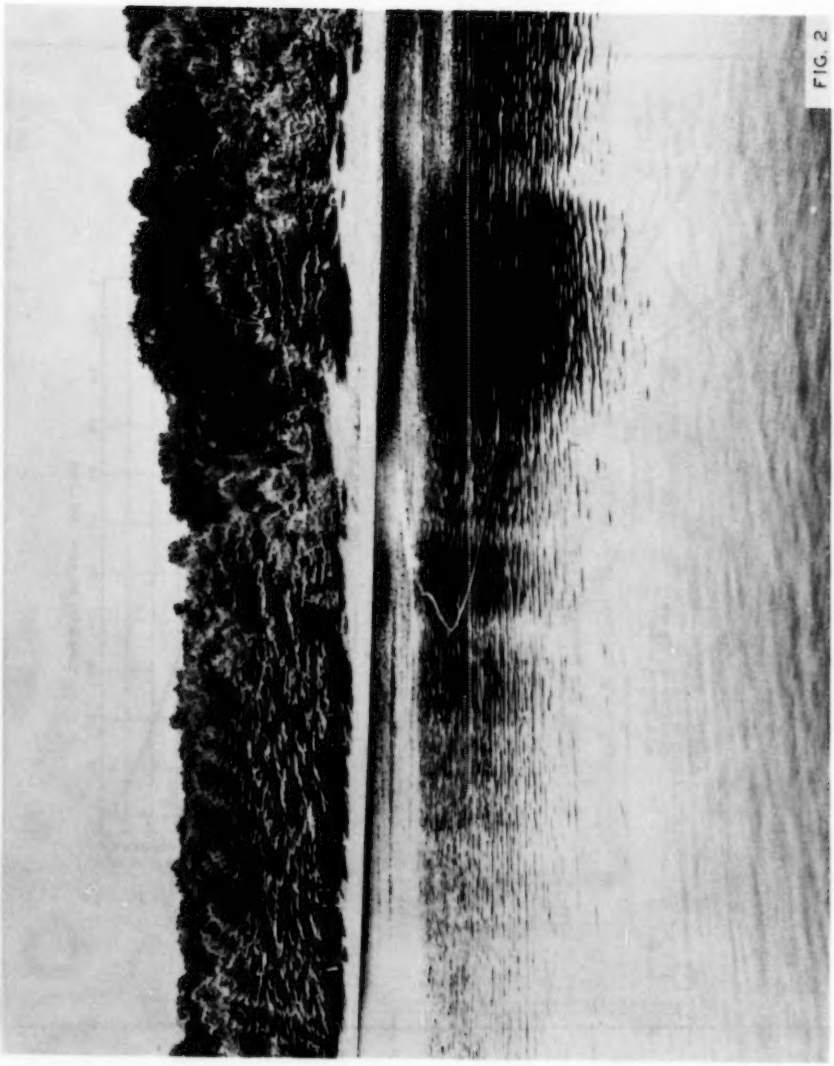
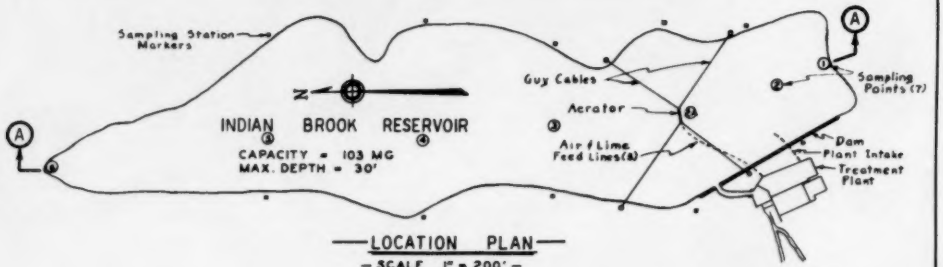
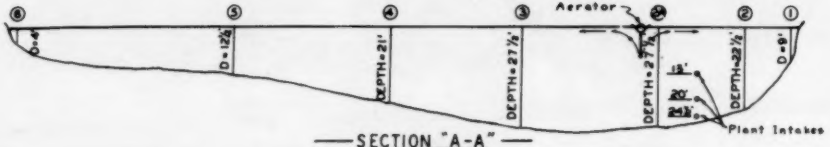


FIG. 2



LOCATION PLAN
SCALE 1" = 200'



SECTION "A-A"
SCALE — HORIZ. 1" = 200'
VERT. 1" = 20'

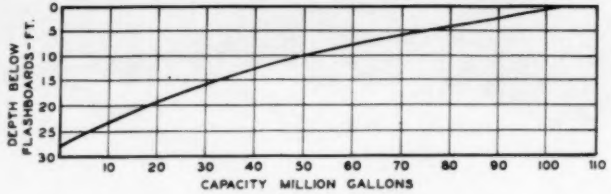
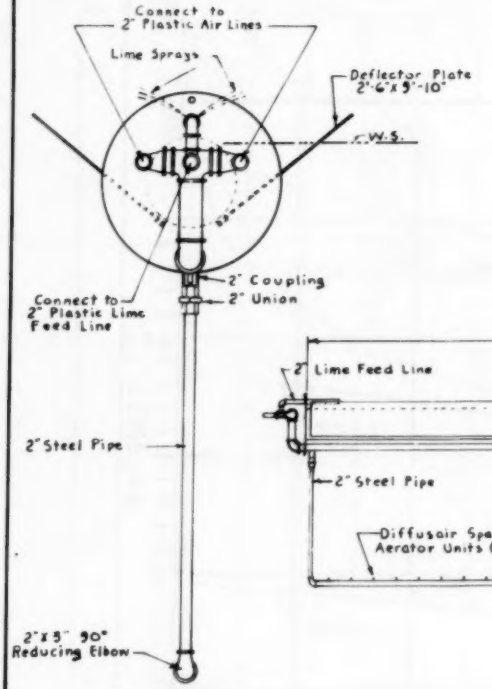
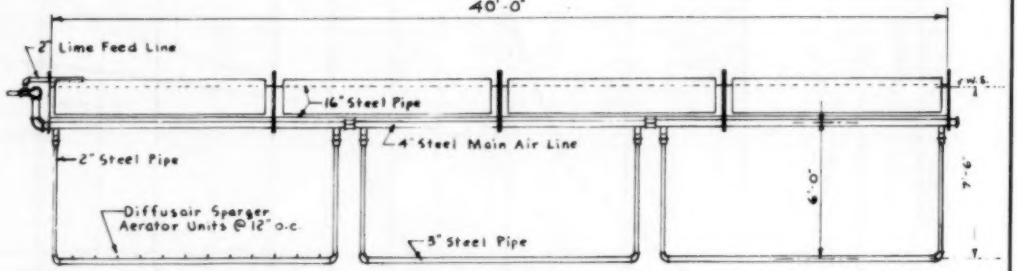


FIG. 3



END VIEW OF AERATOR UNIT

SCALE 3/4" = 1'-0"



ELEVATION

SCALE 1/4" = 1'-0"

FIG. 4

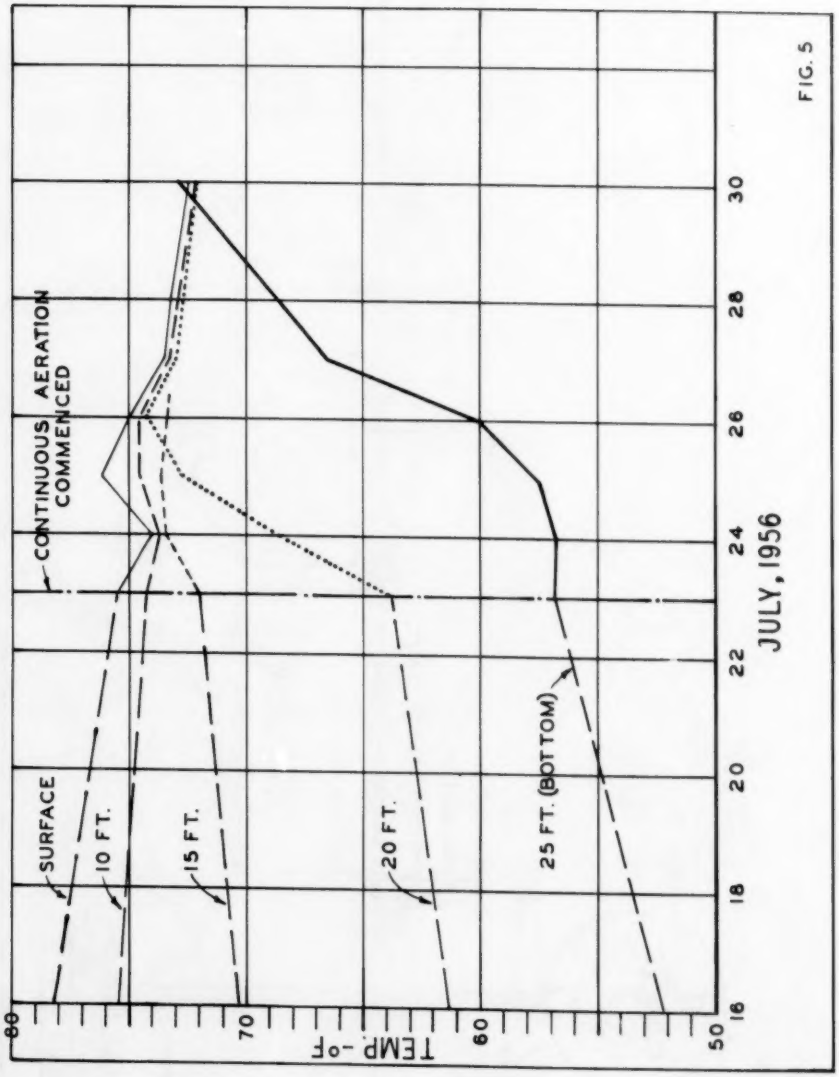

CONTINUOUS AERATION COMMENCED
SURFACE
10 FT.
15 FT.
20 FT.
25 FT. (BOTTOM)
TEMP. -°F
80
70
60
50
16
18
20
22
24
26
28
30
JULY, 1956

FIG. 5

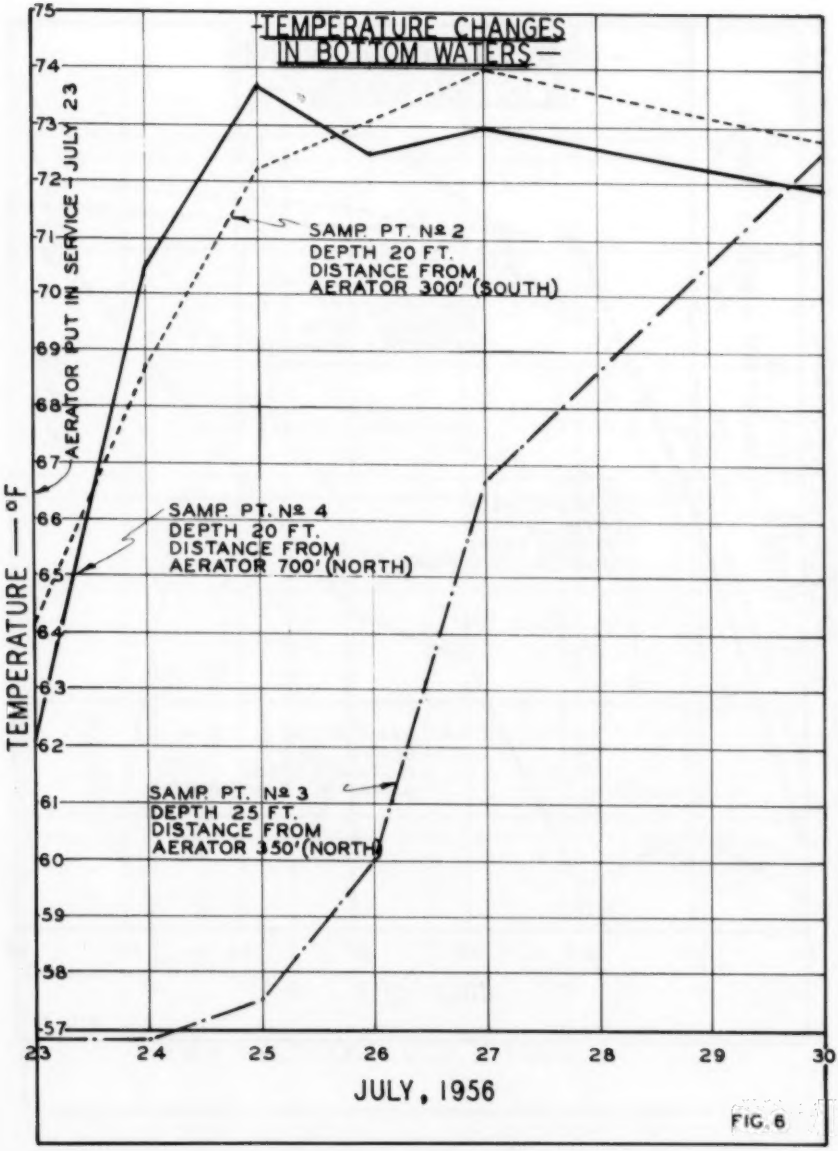

TEMPERATURE CHANGES IN BOTTOM WATERS
AERATOR PUT IN SERVICE JULY 23
SAMP. PT. Nº 2
DEPTH 20 FT.
DISTANCE FROM
AERATOR 300' (SOUTH)
SAMP. PT. Nº 4
DEPTH 20 FT.
DISTANCE FROM
AERATOR 700' (NORTH)
SAMP. PT. Nº 3
DEPTH 25 FT.
DISTANCE FROM
AERATOR 350' (NORTH)
TEMPERATURE — °F
75
74
73
72
71
70
69
68
67
66
65
64
63
62
61
60
59
58
57
23
24
25
26
27
28
29
30
JULY, 1956

FIG. 6

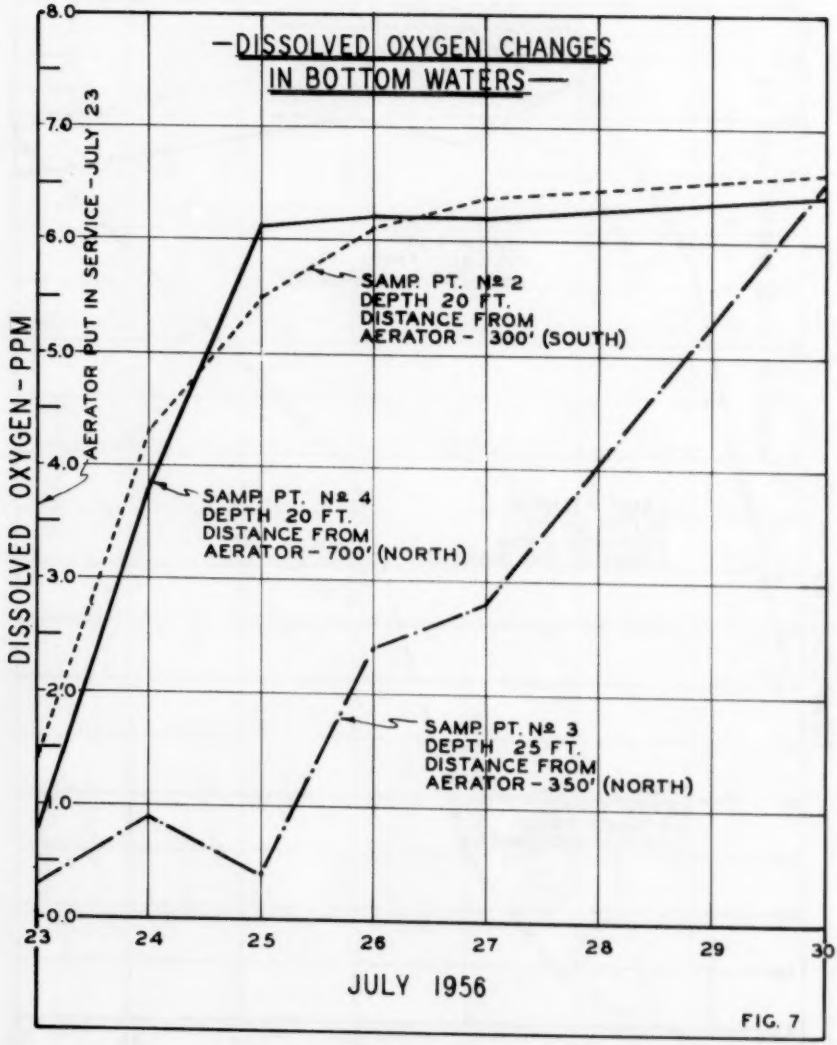

—DISSOLVED OXYGEN CHANGES IN BOTTOM WATERS—
DISSOLVED OXYGEN - PPM
AERATOR PUT IN SERVICE - JULY 23
SAMP. PT. Nº 2 DEPTH 20 FT. DISTANCE FROM AERATOR - 300' (SOUTH)
SAMP. PT. Nº 4 DEPTH 20 FT. DISTANCE FROM AERATOR - 700' (NORTH)
SAMP. PT. Nº 3 DEPTH 25 FT. DISTANCE FROM AERATOR - 350' (NORTH)
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
23
24
25
26
27
28
29
30
JULY 1956

FIG. 7

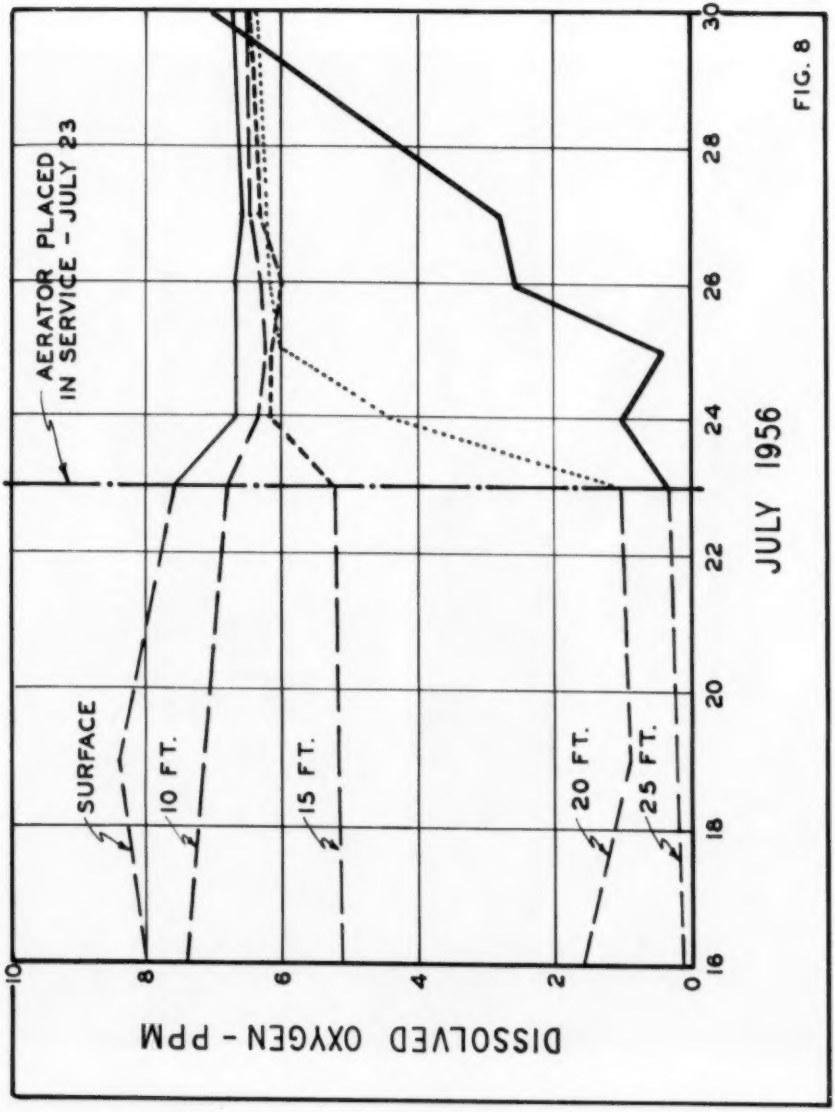
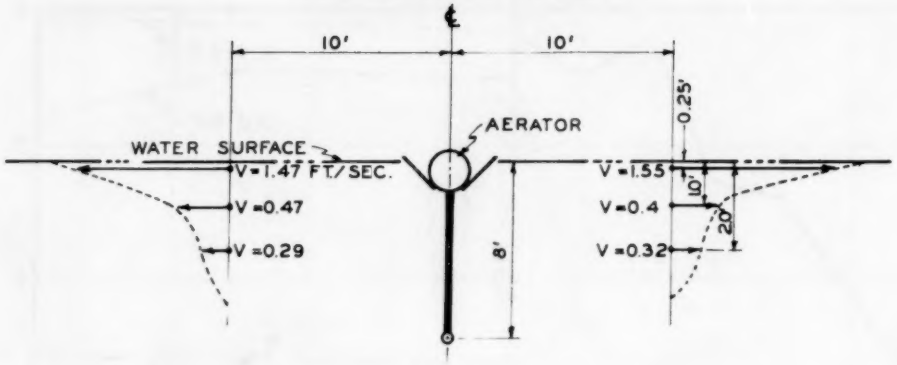

DISSOLVED OXYGEN - PPM
0
2
4
6
8
10
16
18
20
22
24
26
28
30
JULY 1956
AERATOR PLACED IN SERVICE - JULY 23
SURFACE
10 FT.
15 FT.
20 FT.
25 FT.

FIG. 8

—VELOCITY MEASUREMENTS TRANSVERSE TO AERATOR—



Q = AV = 40 X 0.5 X 1.47 = 29.4 CFS
40 X 1.5 X 0.97 = 58.2 CFS
40 X 2.0 X 0.38 = 30.4 CFS
118.0 CFS

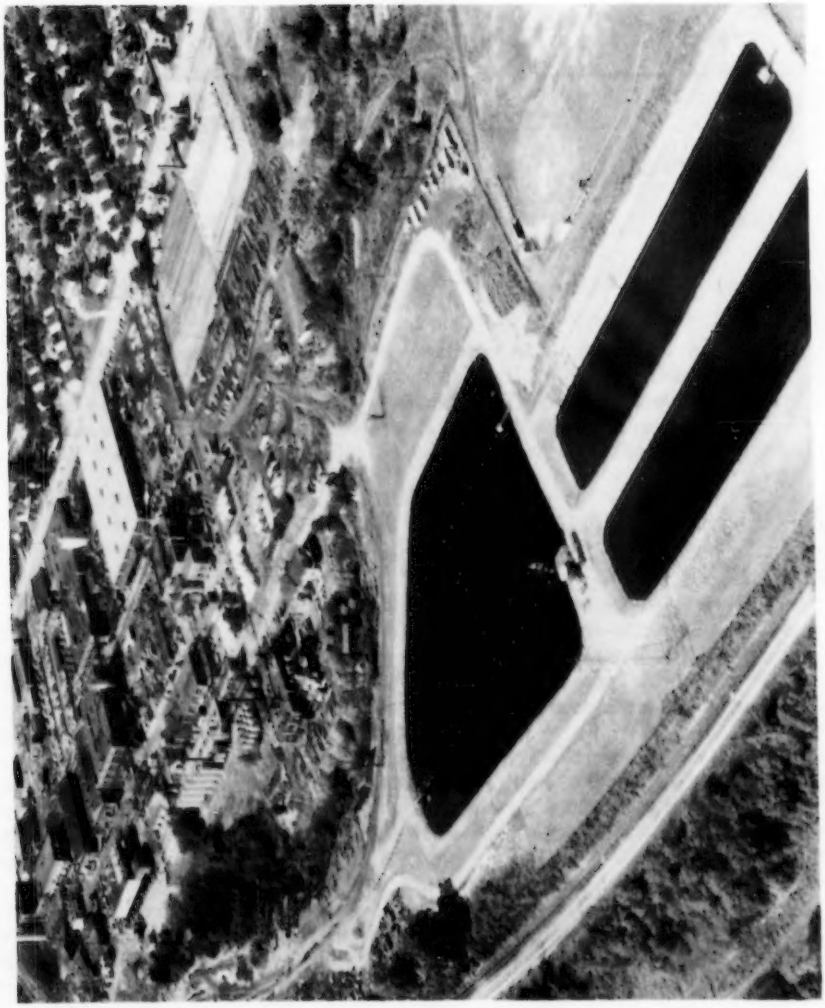
Q = AV = 40 X 0.5 X 1.55 = 31.0 CFS
40 X 1.5 X 0.97 = 58.2 CFS
40 X 2.0 X 0.36 = 28.8 CFS
118.0 CFS

Q = 236 CFS = 153 MGD = 107,000 GPM

WHP = 0.000253 X FT. HEAD X GPM

FT. HEAD = $\frac{8.0}{0.000253 \times 107{,}000}$ = 0.30 FT. = 3½ INCHES

FIG. 9



Paper 1704 July, 1958 SA 4

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

AN INTRODUCTION TO SOLAR DISTILLATION

Tim de Jong,[1] M. ASCE
(Proc. Paper 1704)

ABSTRACT

This paper presents a study and the results of experiments on the production of potable water with solar energy on three pilot plants in Iran. The effects of geographical location, the size and shape of plants, together with descriptions of various plants and methods of distillation, on the actual production of the pilot plants are explained in the report, with charts and tables.

Purpose of this Experimental Project

Heat from the sun has long been considered the future potential energy for the world. It is known that the present sources of energy—oil, coal and water power—are gradually decreasing, with the exception, perhaps, of the latter. Although scientists have recently discovered atomic energy, this source of power is difficult and dangerous to control at the present time; therefore, increased attention has been given to solar energy.

In 1955 George O. G. Lof made a study of solar distillation of sea water for the United States Department of Interior. In his report he estimates "That the total solar energy received by the earth is over 2000 times the energy now being utilized in other forms, but the energy is so diffuse that the quantity reaching any small area is very small."(1)

Lionel S. Marks, in the Mechanical Engineers Hand-Book, states that "the heat received per minute from the sun on one square foot of surface, normal to the sun's rays above the atmosphere of the earth, amounts to 7.12 BTU (British Thermal Unit) per square foot, but before reaching the earth, where it can be utilized, part of this heat is absorbed by water vapor and dust suspended in the atmosphere. In vast areas in the tropics, and in certain

Note: Discussion open until December 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1704 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 4, July, 1958.

1. Chief Civ. Engr., Litchfield, Whiting, Panero, Severud and Associates, Cons. Engrs., Tehran, Iran.

semi-tropic and arid sections (i.e., Arizona, Egypt), however, the air is dry and clear. This loss is small and the sun power can be profitably produced if coal is very expensive and the cost of the land area for the plant is low."(7) In brief, in areas where the need is urgent, the land is cheap, and there are many hours of sunshine, the use of solar energy is practical.

In certain areas in the United States the ground water table and surface water are decreasing gradually, making the land area increasingly more difficult to sustain life. The U. S. Department of the Interior, the Reclamation Bureau, the Weather Bureau, and various state agencies are increasing funds for the study of water conservation.

The U. S. Navy, during the last world war, found that one of its many difficult problems was that of obtaining potable water for the troops stationed in the South Pacific islands. Realizing an even greater need in the future, the Navy is constantly studying methods, mainly solar, to acquire potable water from salt water more efficiently and at less cost.

In the southern third of Iran, where the terrain is largely desert or barren rocky mountains, the inhabitants use, if available, perennial streams as their water source. However, recently the practice of well-drilling has given some relief to a few of the isolated communities. Generally, the water from these wells, although pure, has a high chemical content in calcium carbonate and chlorides, commonly called salts. The calcium carbonates or equivalent in the water cause "curdling" of the water when soap is used, an increased consumption of soap, and deposition of scale in boilers. This characteristic of the water is hard and, although it is not detrimental to health, it does make washing a difficult task. If the chlorides are present above 250 ppm (parts per million), they are detrimental to health and the water is not potable. One method of removing the chlorides is by ionic process, or electrodialysis. Some installations, usually oil refineries, distill sea water for their own use by this ionic process. For this purpose, fuel from the oil refineries is readily available at low cost. An example of this: For generations the small island of Aruba in the Netherlands Antilles has battled a chronic water problem. But 20 years ago the Lago Oil Refinery decided to do something about it. Through the years, after many changes in distiller design, the Aruban Government is now putting ten million dollars into the world's first big-scale salt water still. Arubans will be the first large community in history to get drinking water entirely from the sea. But this process is not practical for small installations—the cost of power required is excessive.

In those isolated areas of southern Iran where there are no perennial streams and where the well-water has too high a percentage of chlorides, villagers gather rain water and store it in anhambars or collecting basins. Sometimes this water is stored for a period of two years, during which time it becomes exceedingly stagnant, and bacilla, bacteria, worms, and other forms of microbic life develop, causing sickness or blindness to the users.

Not only is the need for a potable water source urgent in southern Iran. Other areas, many of them in the United States, face the same problem. In the Christian Science Monitor of February 24, 1957, a news item with pictures appeared. This is quoted in part:

"$1.25 Buys 5 Gallons of Water in Dry Dallas"

"For decades, Dallas (Texas) depended on a municipal lake for its drinking water. Then came the drought, which is now extending into its seventh year.

Lake Dallas has all but gone dry. . . . Distillation plants have sprung up over the city and the precious fluid in the big demijohns (5 gallons of water selling for $1.25 with $1.50 deposit on the bottle) is being quickly snatched up by those who can afford the price. Hydraulic engineers recently made a perfectly serious proposal to lay a giant conduit from Texas to the Great Lakes. They figured on receiving some Federal aid. . . . Dallas' city fathers have decided, however, to tap the Sabine River, separating Texas from Louisiana, for its future water supply. This will require a pipeline 200 miles long. . . ." And Texas is right next to the Gulf of Mexico—

This need for a potable water supply, for people in isolated areas where no other source of energy is available economically, prompted this study to obtain pure potable water economically from existing impotable sources.

Hydrologic Cycle

The movement of water as it relates to the earth is called the "hydrologic cycle." The cycle consists of the movement of water (from the sea, for a convenient starting point) through the processes of evaporation, transpiration, condensation, precipitation, interception, percolation, and runoff. Each of these water transformation processes may occur in various complicated forms.

When sufficiently heated by solar energy or other means, water in a liquid state passes into a gaseous state. This phenomenon is called "evaporation." Atmospheric moisture consists of water vapor, clouds or fog. Water vapor is the gaseous state of water and is present in the atmosphere because of evaporation processes on land or water surfaces. Clouds and fog are created by the condensation of water vapor upon nuclei in the atmosphere, such as dust particles.

One of the basic functions in the life processes of vegetation involves the process of taking water from the soil through the roots, utilizing it in producing growth and maintaining life, and then discharging it through pores into the atmosphere as water vapor. This process of returning soil moisture to the atmosphere is called "transpiration."

When water vapor in the atmosphere is cooled, condensation results; and when the resulting water droplets obtain a sufficient size, they fall as rain. If the raindrops pass through zones of temperature below freezing, hail results. If condensation occurs at temperatures below freezing, snow is formed. If condensation of water vapor takes place directly on a surface cooler than the air, either dew or frost is formed, depending on whether the temperature at which condensation occurs is above or below freezing.

Some precipitation is evaporated during its fall. Some precipitation is intercepted by vegetation, from which a part is subsequently evaporated into the atmosphere. This is called "interception."

Of the precipitation that reaches the ground, some infiltrates into the ground, some runs off over the surface, and some evaporates or is transpired back into the atmosphere.

When the rate of precipitation exceeds the rate at which water may infiltrate into the soil, surface runoff occurs. On sloping terrain with a thin soil mantle, infiltrated water may return to the surface through lateral movement. This is known as "subsurface runoff."

A portion of the precipitation penetrating the ground surface as infiltration

will percolate into the ground and eventually reach a level which is completely saturated, called the "ground water table." The ground water body may intersect a stream bed where part of it will be returned to a body of surface water.

That portion of water which is not evaporated from the land or transpired from plant life will follow paths of various lengths and complexities, and discharge the water into the sea from which the cycle began.(6)

Forces of Nature Affecting Solar Distillation

The same major forces of nature that affect the hydrologic cycle of the earth have an effect on the process of solar distillation. The main difference is the size of the project and the isolation of certain phenomena from the cycle.

The hydrologic cycle is an immense natural phenomenon which does not play favorites in the disposition of its moisture. In solar distillation, man attempts to copy the hydrologic cycle so that he may control and use the returns in his constant quest and search for water in nature's forgotten arid areas.

The solar heat rays are transmitted through transparent membrane, glass, into an enclosed compartment containing the salt or brackish water. Heat from the rays is absorbed in the raw water; and temperature of the water is raised to an evaporating point. The evaporated water, water vapor, rises by convection and saturates the air. The outside temperature cools the transparent membrane, glass, and the water vapor condenses on the inside of the pane. The distillate runs down the pane by gravity, is collected in a trough and is conveyed into a receiving receptable as pure water.

Thus the major forces in the hydrologic cycle—radiation, evaporation, convection, condensation, and dew formation—are utilized.

Source of Solar Energy

A brief summary of pertinent scientific factors relative to solar energy is included in the following paragraphs:

Solar Radiation

The sun gives off energy in the form of radiation. In studies of solar energy uses on the earth, the important general aspects of solar energy may be subdivided and summarized as follows:

a. The Solar Constant

The solar constant is a measure of the intensity of the incoming solar radiation at the earth's mean distance from the sun, normal to the sun's direction. It has been measured as 1.94 Langleys (each Langley is one gram-calorie per square centimeter of surface area) per minute. The solar constant fluctuates slightly due to the earth's orbit and certain unknown causes. For convenience, engineers use the BTU (British Thermal Unit) system of heat units, as 3.69 BTU per square foot equals one Langley.

b. Solar Radiation Intensity at the Top of the Atmosphere

Values for the solar radiation per day on a horizontal surface of the earth have been computed, taken from angles of the sun and the value of the

radiation, along with the change in latitude and season. These values are shown in Chart No. 1. The values cannot be used directly, because of the many energy losses in the atmosphere. These values are shown mainly to indicate the variation of radiation over the earth.

c. Effective Solar Radiation at the Earth's Surface
Incoming solar radiation suffers depletion by:

1. Absorption by ozone in upper atmosphere.
2. Scattering by dry air.
3. Absorption, scattering, and diffuse reflection by suspended solid particles, and
4. Absorption and scattering by water vapor.

Loss of solar radiation in the atmosphere is not a constant, but is dependent on the effect of the land, etc.; thus, actual radiation cannot be obtained by taking a fixed percentage of the radiation atop the atmosphere.

d. Available Records of Solar Radiation
About 18 meteorological stations in the United States have values of hourly radiation on a horizontal surface. Thus meteorologists have been able to compute the average radiation which could be expected at any latitude on any date.

The values usually include both diffuse and direct radiation. They may be used without adjustment for angle of incidence since the water layer for solar distillation is horizontal. For any location, there must be only a rough approximation, because of the wide variations in land, seasons, and latitudes, and the availability of so few observation stations.

In 1950 J. Gabies evaluated the monthly average daily effective solar radiation for different latitudes by 5^{o} increments of latitude, and from this table we have derived a Solar Efficiency Chart for 36^{o} latitude (Tehran) as shown in Chart No. 2.[14]

The most likely place for solar distillation is the area with the least clouds, and the least water vapor in the atmosphere. In such an area, the scientist or engineer may take advantage of full radiation with the least interference.

Methods of Solar Distillation

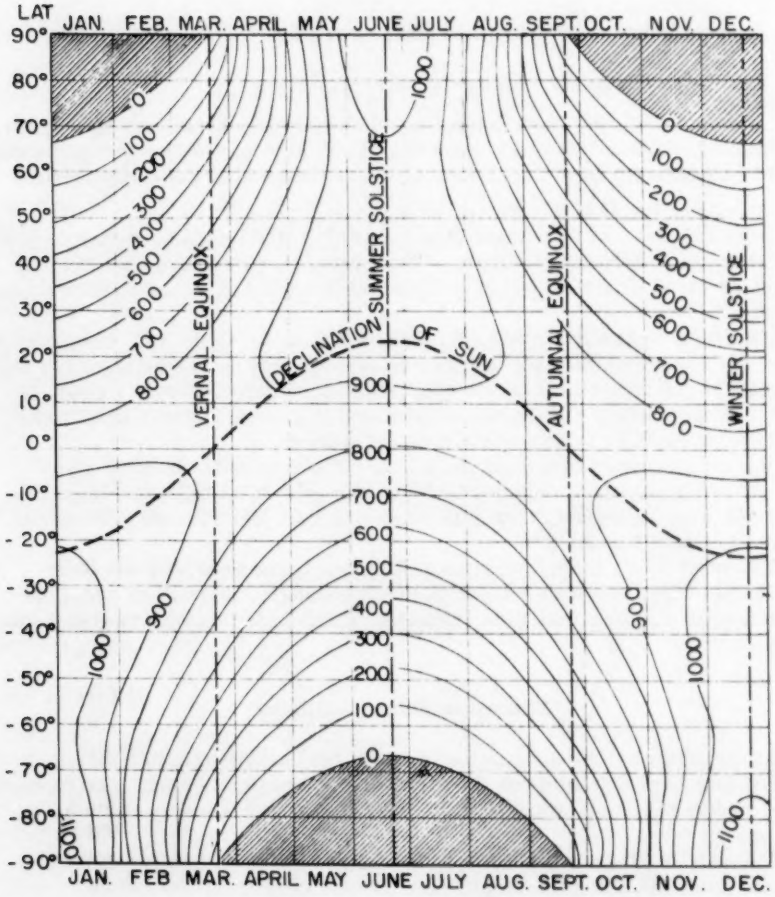
As the idea of solar distillation is fairly new, all the possible forms of pilot plants have neither been invented nor put into experiment. The following types pretty well sum up the variety that have been invented over the past few years:

The different types of solar distillers may be divided into two main categories, those including the employment of electric energy generated in solar power plants, and those using heat developed by a solar energy absorber. The system of electric energy run by solar power would be much more expensive than the present day fuel-burning plants, so this type would be totally impracticable and uneconomical.

In the use of solar heat energy, the type most attractive would be the evaporator-type, which can also be divided into two main groups: First, using the steam produced by a solar energy absorber in a water distilling plant with several effects, or, second, the plants concerned with absorbing the solar

CHART NO. 1*

Chart of the total daily solar radiation at the top of the atmosphere



The solid curves represent total daily solar radiation on a horizontal surface at the top of the atmosphere, measured in cal. cm.$^{-2}$. Shaded areas represent regions of continuous darkness. The solar constant J is assumed to be 1.94 cal. cm.$^{-2}$ Min.$^{-1}$

* Taken from Table 134, Smithsonian Meteorological Tables, 6th Edition, Smithsonian Institution, 1951.

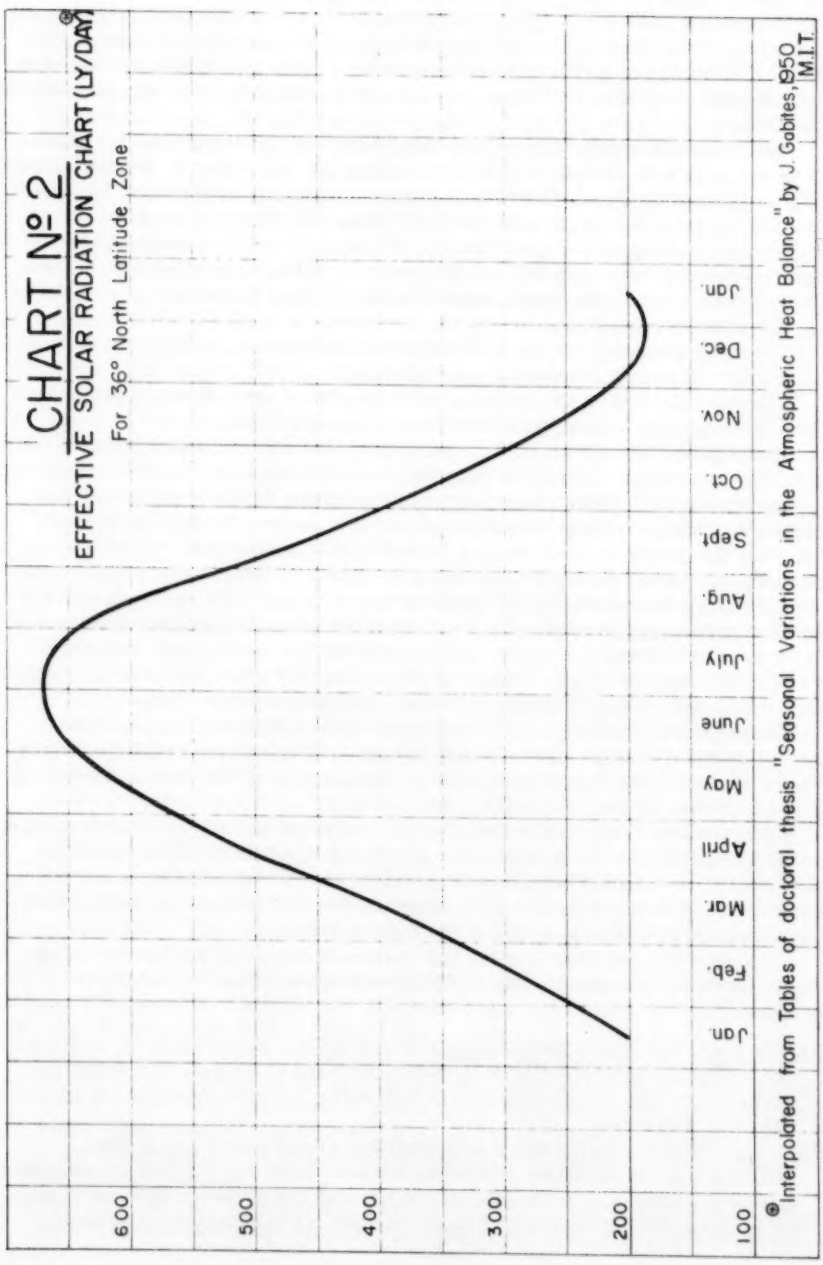


CHART Nº 2
EFFECTIVE SOLAR RADIATION CHART (LY/DAY)*
For 36° North Latitude Zone
600
500
400
300
200
100
Jan.
Feb.
Mar.
April
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.
*Interpolated from Tables of doctoral thesis "Seasonal Variations in the Atmospheric Heat Balance" by J. Gabites, 1950 M.I.T.

energy and evaporating, all in one piece of equipment. Of these types, the most effective for experimental testing would be the simplest in construction, or that of using all energies in one piece of equipment.

To absorb heat, two methods are effective: (1) for high temperatures, the focusing- or mirror-type collector, and (2) for moderate air temperatures, the flat-plate type. The collector-evaporating types can be further divided into those having self-contained condensers and those having separate outside condensers.

The collector-evaporating types are illustrated as being shallow, glass-covered pans with simple devices for putting the salt water in and for collecting it after distillation. This small type has formerly been tested in Chile and recently in the Virgin Islands, California, and Massachusetts, and this report depicts results of operations in Tehran, Iran. This seems to be the least expensive type of distiller. Evaporation units made of plastic in these same designs have been experimented with also; but these have proved to be the least productive types.

The most productive type of distiller seems to be the compression-type, although it is the most expensive per foot.

The partial-vacuum would seem to be practical since it would reduce the loss of both water vapor and solar energy through the air.

Saline water demineralization being used on a commercial basis is divided into two general methods, both involving evaporating a portion of the water by steam from a fuel-fired boiler, and condensing the vapor by means of cold condenser water. One of these processes is a series of evaporation units whereby the vapor of the first in succession is condensed in the coils of the second, and so on, having a complete line of succession, thereby taking advantage of all the material. This would result in as much as five times the amount produced in a single evaporating unit. The other of these processes is by recompressing the vapor, and condensing the vapor in the evaporator itself. The biggest energy of this is, of course, the input of the motor driving the compressor. Only a small bit of the energy is used as steam heat.

These two types of distillers have produced several hundred thousand gallons per day. Ships employ these for general use and to feed the boilers. Vapor-compression types were used in the last war by the Armed Forces to give the troops on land pure water to drink.

Several other types are in research or development stages. Some of those being investigated are the chemical regeneration, electrodialysis through permselective membranes, supercritical pressure reparation, and a few others. They do not compete with compression distillation, but improvements can change the operation cost and the type of distiller.

Following are explanations for the various methods of distillation equipment, listed to correspond with the Figure Number at the end of this section:(1)

Figure 1—Compression distillation, which need not necessarily be called a solar demineralizer, because it would not have to be part of a solar installation. Electric power—either hydraulic, thermal or solar—would be used for this type.

Figure 2—"A process for sea water distillation and power production, utilizing the temperature difference between deep and surface ocean water, is being installed in West Africa; the normal temperature difference will be increased a few degrees through passing the surface water through

shallow, solar-heated ponds prior to evaporation. Vacuum evaporation, followed by condensation in units cooled by the cold water from the depths, will complete the process."

Figure 3—A typical three-effect conventional evaporation unit heated by steam produced in a solar boiler, and of the common submerged tube type. The water vapor from the first container circulates through tubes in the second container, causing the formation of water vapor, and so on. In the last-stage condenser, the heat is exchanged from the condensate to the sea water, preparing it for the containers in which it will change into water vapor.

Figure 4—A typical design of the type of evaporator using steam generation in a flat plate collector. "The blackened metal plate, beneath three glass plates, is the heat absorbing surface. The water is heated under pressure in the tubes attached to the metal plate, and after flashing," the result is steam, which may be used in a "conventional multiple-effect evaporator system."

Figures 5, 6, 7, and 8 are illustrations of conventional evaporators which are extensively modified for effective utilization of energy from a separate solar collector. There are several types of these, one of which is shown in Figure 5.

Figure 5—A three-effect evaporator, which is used to heat sea water by means of solar-heated air. The air goes through the solar collector (made up of a series of overlapping, partially-blackened plates) into a series of temperature exchangers which preheat the sea water. At the last temperature exchanger, the air is cool, and is transported back up to the solar collector to go through the same cycle.

Figure 6—Solar-heated air is used to vaporize preheated sea water by a spray changer. The vapor then goes down a pipe into a condensing chamber, where the condensate drops down, and the cool air is transported up for reheating.

Figure 7—The sea water is preheated in three exchangers, some is evaporated by pressure in a focusing collector and some steam is expanded through a turbine which drives a vapor condenser. The steams are condensed by being mixed with cooler distilled water. All the condensate is used as a heat supply to an evaporator (partial vacuum). Vapors are then sent to the compressor, thus being solely operated by solar energy.

Figure 8—The sea water in Figure 8 is preheated in several exchangers. It is then partially vaporized in a spray chamber by a jet of preheated air. Air vapor mixture is compressed slightly and heated in a focusing solar collector. Then it is expanded through a hot air turbine connected to the compressor. The mixture is cooled in the exchangers and pure water condenses from the air.

Of the equipment with solar energy collection and water evaporation in a single unit, Figures 9 and 10 are of the multiple-effect type and Figures 11 through 17 are of the single-effect type.

Figure 9—An illustration of the type originated for users of life rafts at sea. Saline water is fed into the evaporator wick layers of the series of assembled porous films and wicks in sheet form. Solar energy absorbed by the black wick causes evaporization of a portion of the water from the wick. It goes through a wick condensation and then drains to storage. This is a continuous process.

Figure 10—The solar energy that passes through the glass sheets is absorbed by a black metal sheet, over which a thin layer of saline water is flowing.

Evaporation into the circulating air, and condensation of vapor on the sheet behind the evaporator sheet. The heat of condensation then evaporates the thin layer of saline water on the next evaporator sheet.

Figure 11—The sea water is fed slowly into one end, and is vaporized into the air enclosed in the unit. Moisture condenses on the sloping glass and collects in channels at the base of the glass. This basic type is very similar to our own Pilot Plant No. 1.

Figure 12—Another form of the basic single effect, one-unit plant, but this is used to intensify the solar energy on the horizontal evaporating pan and would be most useful at high latitudes.

Figure 13—An illustration of a black evaporating tray in a tubular glass unit, having the same reaction as Figure 11, with a greater proportion of condenser surface to evaporator surface.

Figure 14—Similar to Figure 13, with exactly the same reaction, except the use of a sheet-assembled tubular rigid plastic unit is made.

Figure 15—This design shows the basic idea of the single-effect, one-unit plant, but it has the solar absorption take place in a black wick saturated with saline water, which is placed directly under the plastic cover. Evaporation into the air takes place and condensation results on the collar wall.

Figure 16—A tubular, plastic case containing a separate black tray with black evaporating wick, to have an over-all covering of saline water, level, or not, is shown. It is actually the same plant as Figure 14 except that rigid sheets are replaced by a single plastic film, wire-reinforced to save material.

Figure 17—A life raft survival kit for aviators contains a semblance of Figure 17, a globular floating plastic still which is inflated and uses a black evaporating wick, saturated with sea water. Evaporation takes place on the evaporator wick; condensation is performed on the cool walls of the plastic material, both on the sides and underneath the horizontal wick.

Figure 18—A modification of the single-effect distiller in one unit (Figure 11) is shown. The vapor-laden air is circulated to an external water-cooled condenser.

Figure 19--The single-effect distiller plant is shown in another variety: the shallow pan covered with a black metal heat-absorbing solar collecting surface. The fins which project down from the metal cover help to direct the radiation to the water, helping to reduce heat losses.

Figure 20—If a black metal tube were used for the inside tube, the dye would be eliminated, as it is used in this Figure 20. The dyed saline water goes in the plastic tube, is heated by the heat inside the outer plastic tube which serves as an insulator, and vaporizes into the air circulating above it. The air is transported to an outer condenser (salt-water cooled) and the air is recirculated into the tube. This cycle is illustrated in all the plants shown on the drawings in this section.

To develop a water supply system in a remote country where machinery and materials are either non-existent or difficult to obtain and where operators are unskilled, a simple type of distiller is required. For these reasons and under these conditions, the plants illustrated in Figures 1 to 10 inclusive and 13, 14, 15, 16, 18, 19, and 20 have not been considered practicable. The plant shown in Figure 17 is too small to produce any quantity except under life-saving emergency conditions. Plants illustrated in Figures 11 and 12 are adaptable for air-cooling condensing type under the remote conditions of Iran.

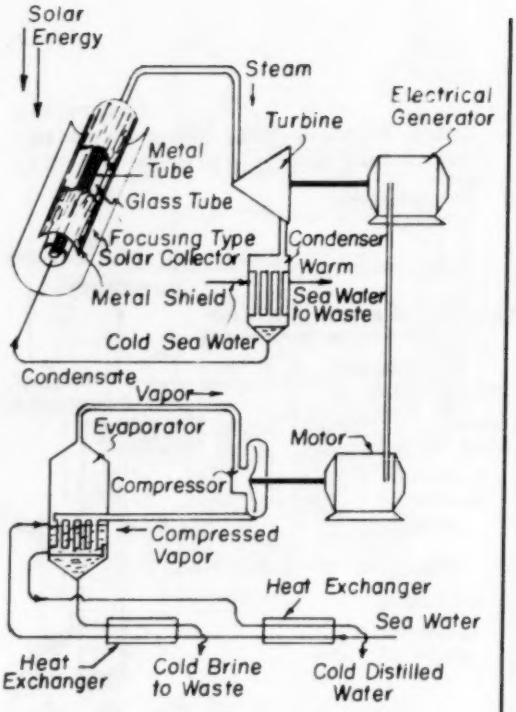


Fig. 1. Compression Distillation Unit Employing Electric Energy from Solar Power Plant.

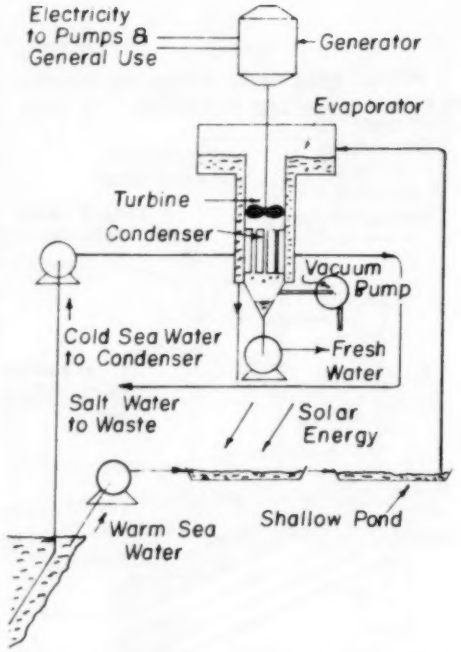


Fig. 2. Electricity and Distilled Water by use of Ocean Temperature Difference and Solar Preheating of Sea Water.

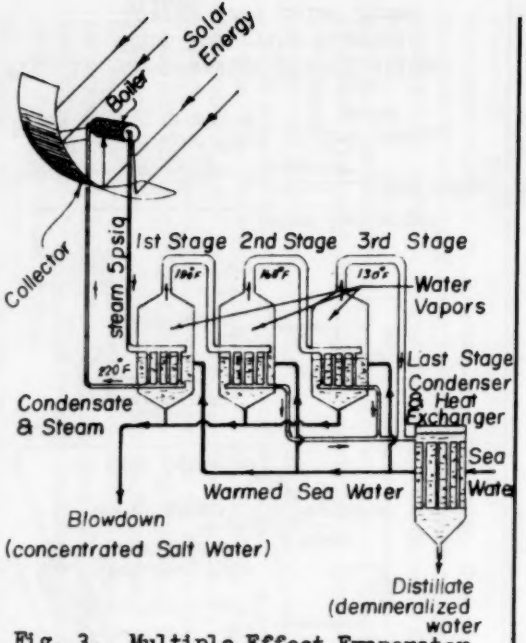


Fig. 3. Multiple Effect Evaporator Heated by Steam from Focusing Solar Collector.

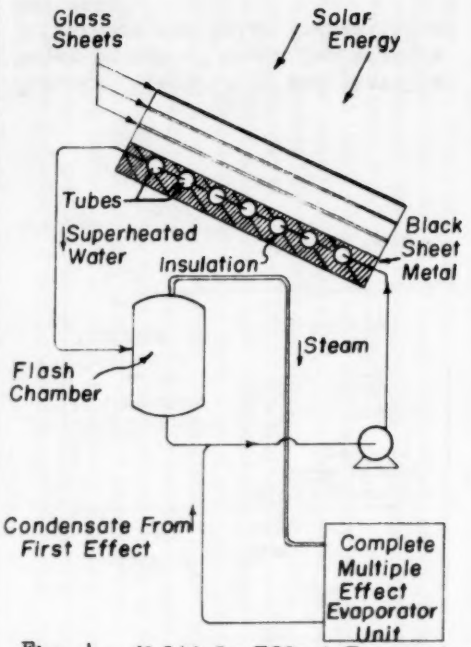


Fig. 4. Multiple Effect Evaporator Heated by Steam from Flat Plate Collector

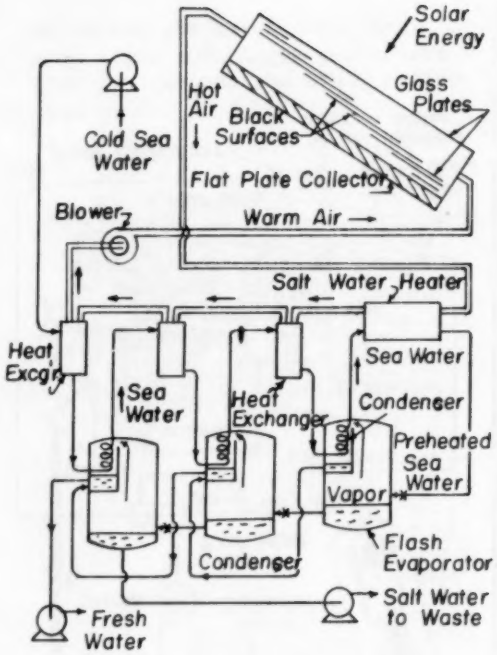


Fig. 5. Flash type multiple-effect evaporator emnloying hot air from overlapped flat plate collector.

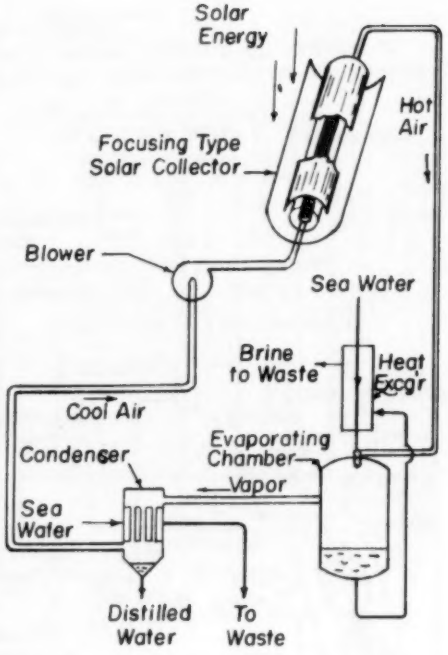


Fig. 6. Atomization and vaporization of sea water with solar-heated air.

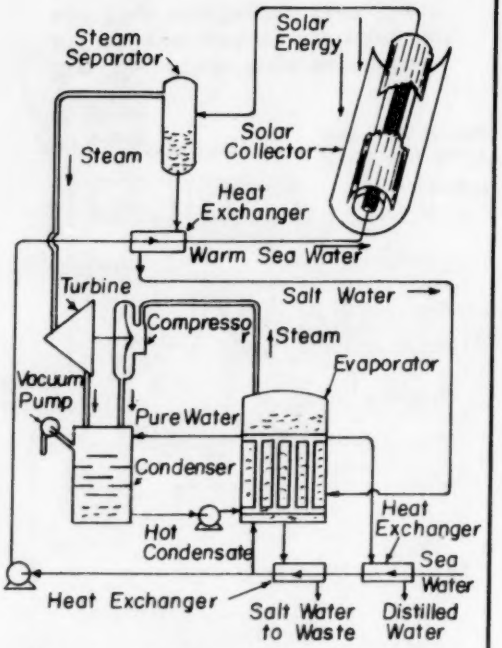


Fig. 7. Compression distillation with solar energy for compressor drive and sea water heating.

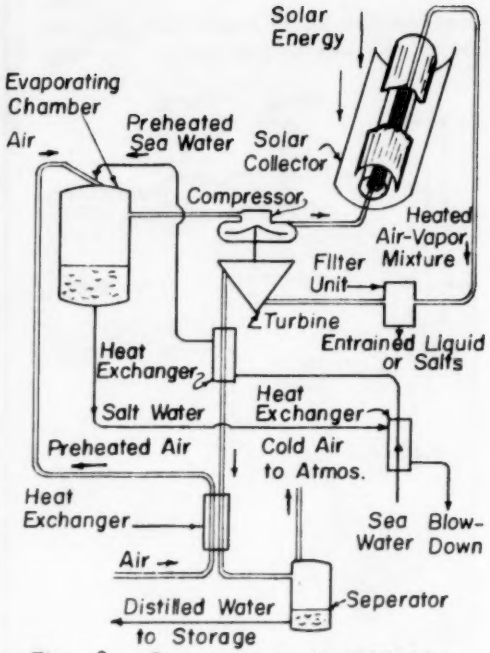


Fig. 8. Compression distillation with solar heating of air-vapor mixture.

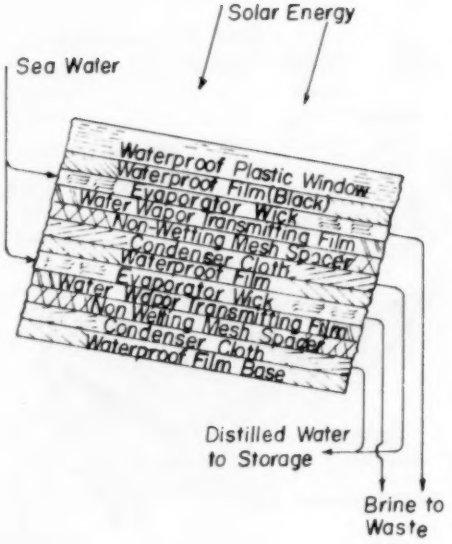


Fig. 9. Multiple-effect evaporation in solar heated, flexible membrane layers.

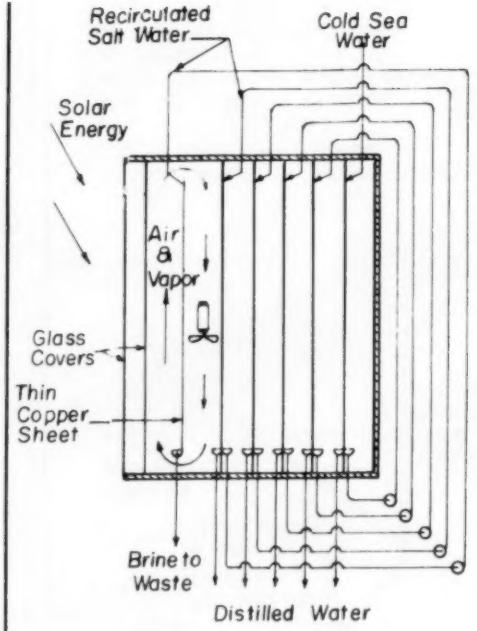


Fig. 10. Multiple-effect solar still employing glass covers and copper condenser-evaporator plates.

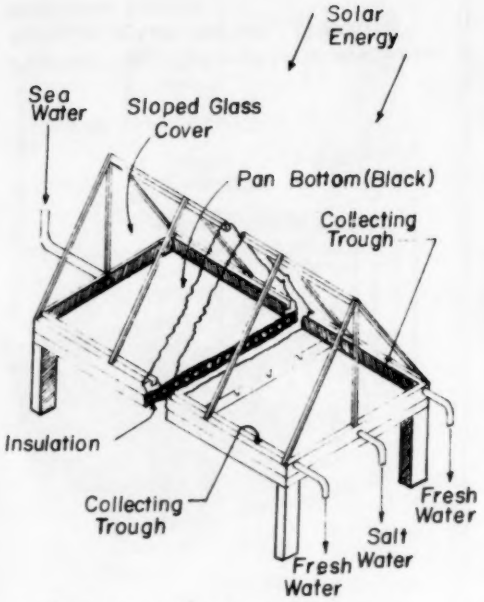


Fig. 11. Glass covered evaporating pan.

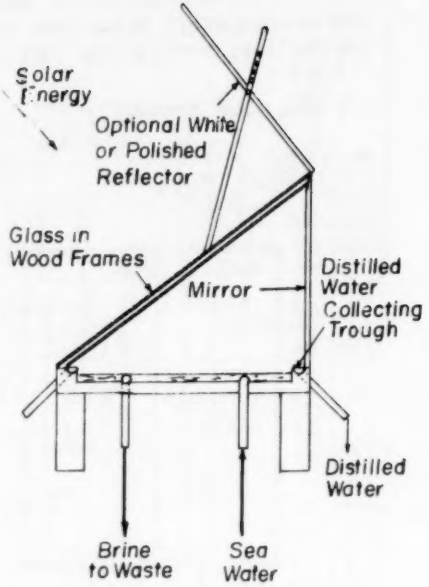


Fig. 12. Glass covered evaporating pan with reflecting surfaces.

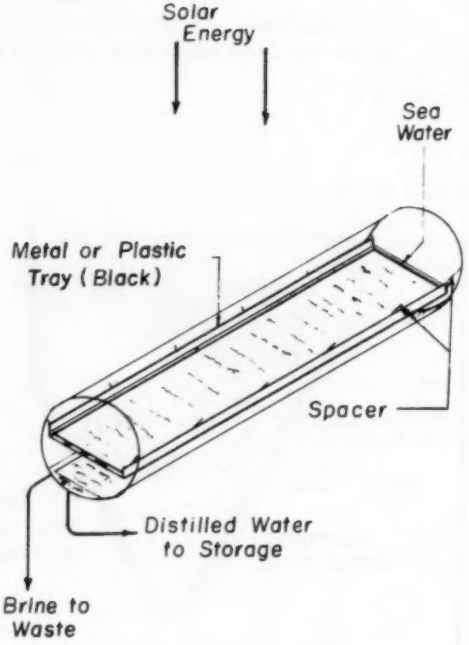


Fig. 13. Evaporating tray in tubular glass unit.

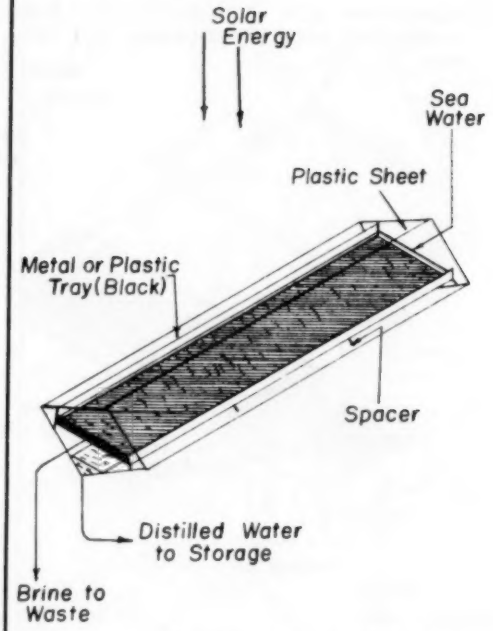


Fig. 14. Evaporating pan in sheet-assembled plastic unit.

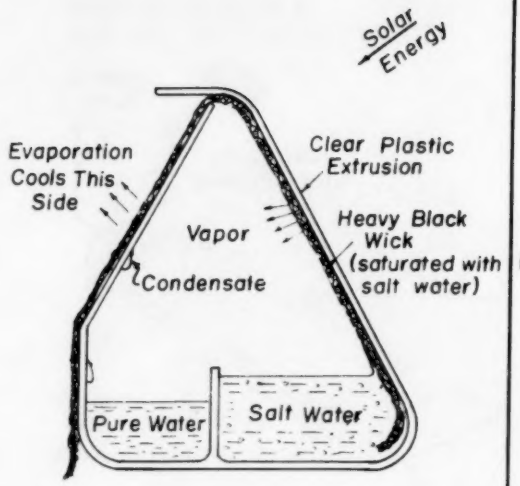


Fig. 15. Extruded plastic still with black wick for evaporation and cooling.

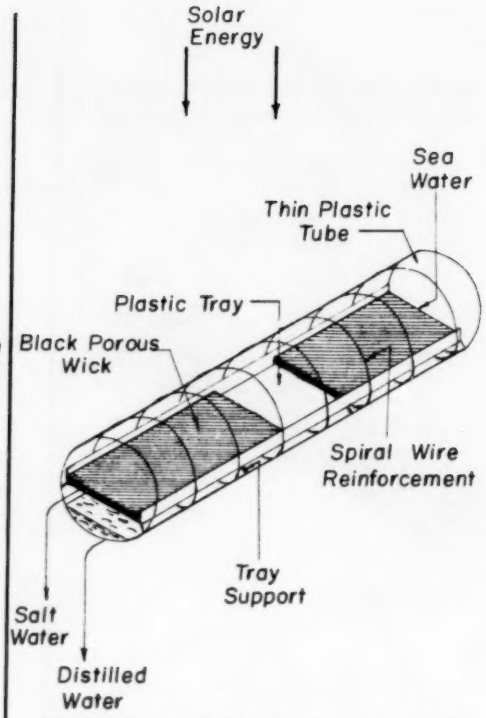


Fig. 16. Reinforced thin plastic tube containing tray with evaporating pad.

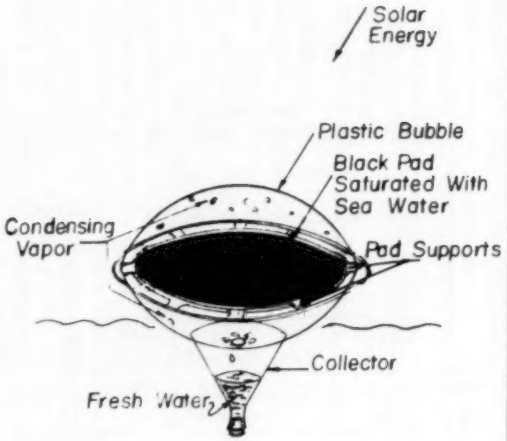


Fig. 17. Floating inflated plastic still with porous black evaporating pad.

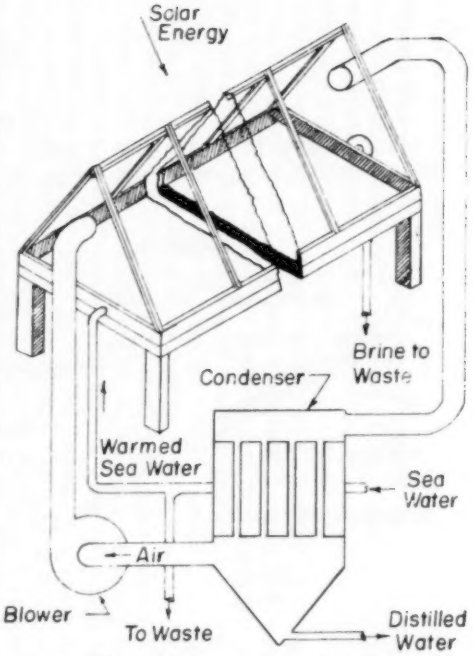


Fig. 18. Glass covered evaporating pan with external condenser.

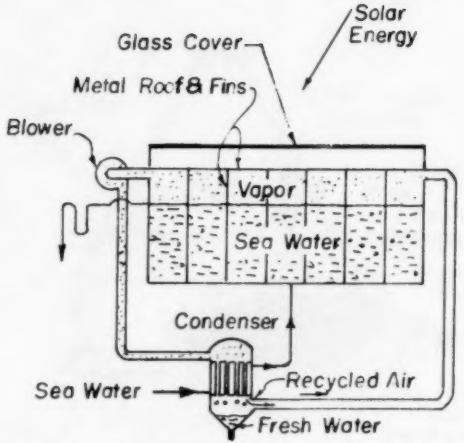


Fig. 19. Metal covered evaporating pan with external condenser.

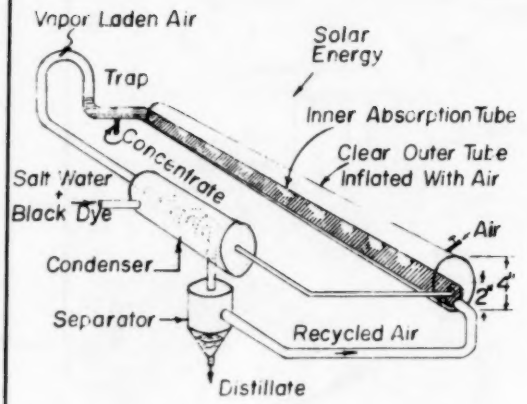


Fig. 20. Single-effect evaporation in double tubular inflated plastic still.

Pilot Plants used in this study are described in the chapter that follows and are pictures on Pages 37 and 38.

Pilot Solar Distillation Plants

Description of Equipment

Three different pilot plants, shown on Pages 37 and 38, have been used in these experiments to determine the most efficient method of solar distillation for a commercial-sized plant at the most economical cost. The designs evolved in a logical sequence, as follows:

Prior to receiving the information shown in Figures 1 to 20 in the preceding chapter, Pilot Plant #1 had been constructed, taking its shape by necessity in order to collect and convey the condensate of the evaporated water.

Pilot Plant #2 was designed to intercept all of the sun's rays. The panes were constructed, as nearly as possible, perpendicular to the sun with the thought of getting the greatest amount of sun's rays over the longest period in the raw water pan; the back was constructed of opaque material to prevent the sun's rays from passing through the plant. This construction, which was believed to have a higher efficiency, lost its efficiency advantage as it was too difficult and expensive to construct for the conditions in Iran. Nevertheless, experimental tests were run on this machine for comparative reasons.

Pilot Plant #3 was constructed with a view in mind to condense the saturated air in coils outside of the plant itself, with the thought to increase the efficiency by preventing the water film from forming on the pane. This water film reduces the amount of sun's rays entering the machine. The available vacuum pump that was used was too small to have any effect, and this process was then abandoned and cooling coils were inserted similar to those in Pilot Plant No. 2. Ironically, this set-up, as shown hereinafter, proved to be the most efficient.

The three pilot plants are described below:

a. Pilot Plant No. 1

The design is a simple air-cooled condenser type. It is designed on a greenhouse-type pattern. It consists of a flat tray containing only sufficient water for one day's distilled water output and losses for each day's operation. The tray is enclosed and covered by an inclined glass roof. The vapor evaporated by the sunlight recondenses on the inside of the glass roof which is being cooled by the outside air; and the liquid then runs down as fresh water into collecting troughs at the side of the tray.

The sloping panes are set within the outer edge of the pan floor to take advantage of any rain which might come, although not considered in this project. The two ends are of tin, similar to the bottom. The north pane, which does not get the sun's ray, is used as a condensate surface. A centigrade thermometer is hung on each pane to determine the difference of temperatures on the north and south panes, and a thermometer is set within the tray to determine the raw water temperature.

The specifications of Pilot Plant No. 1 are as follows:

1. Tray area	16" x 28"	3.10 sq. ft.
2. Height of ridge above tray		21.5 in.
3. Glass area	2 x 19" x 25.5"	970 sq. in.
4. Volume	19" x 25.5" x 21.5"	4816 cu. in.

5. Frontage (transparent)	26 in.
6. Slope of glass to horizontal	68.5°

b. Pilot Plant No. 2

This design is based on constructing the glass panes over a metal tray, similar to Plant No. 1 except that the glass panes are constructed in sections on three sides of the plant.

Plant No. 2 has six panes, unequal in size, each perpendicular to the sun at a particular time of day and, in general, forming a semi-circle facing east, south and west. The purpose is to get the fullest possible use of the sun for the entire time it is above the horizon. All panes are made of glass in triangular shape to enable them to come to a focal point at the top. The largest side, the back which faces north, is made of galvanized tin as are the evaporating pan and the collection channels. One centigrade thermometer measures the temperature of the water inside the machine; another measures that of the pane. A second step to increase the efficiency was introduction of a water-cooling area on the north face of the plant. The purpose was to eliminate the thin film of water on the transparent pane as this film reduces the intensity of solar rays, for condensing surface, on the evaporating pan. Another purpose was to obtain a cooling medium more constant and of greater extent than the erratic weather temperature.

The water-cooling coils on which the water vapor in the plant condenses are installed within the plant. The condensed water is collected in troughs and conveyed to a flask receptacle.

The specifications of Pilot Plant No. 2 are as follows:

1. Tray area	2.50 sq. ft.
2. Height of ridge above tray	11.5 in.
3. Glass area	260 sq. in.
4. Volume	1833 cu. in.
5. Frontage (transparent)	15.5 in.
6. Slope of glass to horizontal	41° 30'
7. Copper cooling coils - length	14 ft.
8. Copper cooling coils - surface	168 sq. in.

c. Pilot Plant No. 3

The design is based on the simplest type of construction to reduce the initial plant cost and to use available local materials as much as possible. It is surmised that possibly brick walls could be substituted for the wood back and end walls used in the construction of this pilot plant.

The raw water receptable is also a flat metal tray similar to Plants 1 and 2. The glass roof is laid on a slope, not necessarily perpendicular to the sun's rays, to conform to the available size of lumber used for the ends and back. It was noted that the angle of the pane to the sun did not affect materially the actual production or efficiency of the other pilot plants. Other factors are important to consider. A series of copper cooling coils is installed inside the enclosed compartment. The distillate collected on the copper cooling coils drips down into a trough and is conveyed by gravity to a glass flask. Necessary thermometers are provided to enable proper records to be kept.

The specifications of Pilot Plant No. 3 are as follows:

1. Tray area	18.5" x 25"	3.20 sq. ft.
2. Height of ridge above tray		12.5 in.
3. Glass area		550 sq. in.

4. Volume $\frac{12.5 + 1.5}{2}$ x 18.5" x 25"	3234 cu. in.
5. Frontage (transparent)	25 in.
6. Slope of glass to horizontal	34°
7. Copper cooling coils - length	21 ft.
8. Copper cooling coils - area	332 sq. in.

Records of Operation

Records of operation have been taken on three pilot plants. As the work progressed, it was found that additional pertinent information was necessary to give data for determining the most efficient type of plant for a given locale.

In the beginning, only weather and water temperatures were taken into consideration in the record of production. It was found that the amount of raw water in the plant affected production. It was found, also, that the temperatures of the condensing surfaces affected most the production. Heat losses from radiation, effective proportion in size of plant and, in the water-cooling condensing plant, the area of coil cooling surface, heat lost in condensation—all had a large effect on the plant production. Therefore it was necessary to take additional data for each of the three plants to indicate comparisons and methods of improving plants for greater production and efficiency.

During the day whenever possible, the temperatures of the water in the distiller (water In), of the weather outside (Out), and of the glass pane of the plant (Pane) are recorded on the hour every hour. All other information, such as weather conditions—whether desert, local, or water-cooling conditions are simulated—and the number of drops per minute (timed) of each pilot plant, is recorded also.

By closing the doors and windows of the greenhouse, where the pilot plants remained during the winter, desert conditions were established. These conditions were used to determine if an air-cooled condenser type solar distiller could be used at all locations.

The water-cooling condenser type solar distiller (copper coils against one pane in these pilot plants—the coils hooked up to a water faucet for convenience) was used to determine the effectiveness of the water-cooling condenser, both in desert conditions and for local conditions of the latitude 36°, or Tehran.

Local conditions were utilized by opening the windows of the greenhouse and letting all conditions—rain, wind, clouds, or snow—affect the process of distillation. The condition determined the percentage efficiency between air and water-cooled condenser for desert and local conditions. This, of course, is only for latitude 36°, or Tehran. Rates change for different climates and different weather conditions.

A chart is made for each day for each pilot plant, and certain formulas are used to derive the proper information for each section (see "Sample Computations," Section VIII). From the temperatures and quantities recorded each day and the charts on the various experiments, proved by the higher institutions listed, the following constants are found:

1. BTU required to evaporate one pound of water.
2. BTU recovered in the plant
   a. distilled
   b. heated but lost

TABLE No. 1

TESTS OF DRIP TRAY EVAPORATION

PLANT No. 1

Summary of Results - Total for month = 4410 c.c. = 9.7#

Date April 1957	Water Temp. °F	Max. Water Temp. °F	Difference in Water Temp. °F	B.T.U. per Lb. of Water Reqd. to Evap.	Returns N c.c.	Returns S c.c.	B.T.U. Recovered Distilled	B.T.U. Recovered Heated	Effective Solar Heat B.T.U.	Installation No.	Efficiency %	Amt. of Water in distiller (left heated) c.c.	Max. diff. in Water and Pane Temp. - °F	Recorded Temperatures of Weather Max. °F	Recorded Temperatures of Weather Min. °F
1	42	106	64	1140	190	165	889	35	5148	13	17.9	250	27	54	35
2														56	28
3														58	34
4	44	47	3	1138	5	5	22	6	5329	13	0.5	990	2	58	36
5	44	80	36	1138	0	40	91	50	5352	13	2.8	960	21	40	34
6	46	76	30	1136	0	50	124	66	5386	13	0.9	1000	14	42	41
7	47	58	11	1135	0	14	44	20	5443	13	1.1	836	3	52	42
8	49	56	7	1133	35	20	136	12	5606	13	2.6	755	6	53	45
9	48	100	52	1134	70	90	398	69	5632	13	8.3	595	23	52	40
10	48	120	72	1134	200	136	839	24	5663	13	15.2	150	32	59	41
11	58	94	36	1124	0	50	124	26	5720	13	3.5	950	22	70	48
12	60	116	56	1122	195	150	842	28	5770	13	15.9	640	25	69	52
13	55	113	58	1127	120	120	537	51	5834	13	11.1	400	22	72	50
14	58	117	59	1124	140	130	674	31	5853	13	12.0	240	22	69	52
15	58	116	58	1124	170	150	787	32	5892	13	13.9	250	25	80	54
16	59	112	53	1123	180	100	673	63	5914	13	12.5	548	25	77	47
17	60	118	58	1122	175	144	785	63	5948	13	14.2	500	20	75	53
18	59	118	59	1123	190	136	786	59	6006	13	14.0	464	27	75	55
19	64	114	50	1118	230	160	1005	45	6028	13	17.4	410	36		
20	64	112	48	1118	78	74	335	82	6063	13	6.9	750	28		
21	60	104	44	1122	116	56	448	56	6086	13	8.4	674	17		
22	59	118	59	1123	85	90	449	94	6120	13	8.8	735	25	76	52
23	58	92	34	1124	45	50	225	58	6177	13	4.5	790	14	75	56
24	58	104	46	1124	175	90	674	69	6212	13	11.9	700	15	67	50
25	61	110	49	1121	150	155	785	28	6246	13	13.0	560	26	65	53
26	60	110	50	1122	140	125	651	65	6280	13	11.4	600	24	68	50
27	57	104	47	1125	120	85	506	74	6315	13	9.2	715	15	68	54
28	58	70	12	1124	10	35	113	28	6350	13	2.2	950	4	71	49
29	59			1123	5	100	258		6372	13	4.0		4	63	51
30	63	94	31	1119	40	100	347	51	6406	13	6.2	750	23	61	46
Ave.	52	99	42	1126	95	52	446	48	5755	13	8.3	597	15	65	47
Max.	64	120	72	1140	230	160	1008	94	6406	13	17.9	1000	36	80	56
Min.	42	Ep	3	1118	0	5	44	6	5148	13	0.5	250	2	42	34

3. Effective Solar Radiation in BTU
4. Efficiency percentage

A comparison is made of all these items for each day and, through this, efficiency can be improved and pertinent conclusions can be drawn.

Sample Computations

Following is the data obtained from the operations of Pilot Plant No. 1 for an average day in early spring--April 1, simulating local conditions:

Water In		42° F.
Maximum Water		106° F.
Difference		64° F.
BTU per pound		1140
Returns:	North	190 cc
	South	165 cc
Quantity left in plant		250 cc
BTU received	Distilled	889 cc
	Heated	35 cc
Effective Solar BTU		5148
Efficiency		17.9%
Maximum Diff. Water and Pane		27° F.
Recorded Weather	Max.	54° F.) Windy,
	Min.	35° F.) cloudy

Following is a sample computation using the above quantities, temperatures, and solar radiation constants to determine the efficiency of the plant:

a. Determination of heat required to evaporate one pound of water at the given temperature:

212° - boiling point of temperature
42° - temperature of water in plant - start of day
170° - temperature rise to boiling point, equals BTU required to raise one pound of water to boiling point, 212° F.
970 - latent heat in water
1140 - BTU required to evaporate one pound of water

b. Determination of amount of heat required to vaporize the amount of water distilled:

355 cc = 0.355 Kg. of water
2.2 lbs = 1 Kg.
0.355 x 2.2 = 0.781 lbs. of water
0.781 x 1140 = 889 BTU

c. Determination of amount of the heat left in distiller and not vaporized. Amount of water--250 cc. divided by 1000 to get Kg. of water, and multiplied by 2.2 to convert to pounds of water = 0.55. The maximum temperature rise of water in plant was 64° F. at 1300 hours. The amount of water required--0.55 x 64 = 35 BTU

d. Determination of Total Heat Generated
If this excess water were not left in the machine, the heat generated in the plant would have been capable of vaporizing additional water. The actual efficiency of the plant must take into account the actual total heat generated. The total heat produced is the heat required as shown in (b) above:

	889 BTU
plus heat left in the plant	35 BTU
Total Heat Produced	924 BTU

e. Determination of the total effective solar radiation. The effective solar heat in Langleys as shown on Chart No. 2 is 450 for April 1 at 36° North Latitude. This is the earth's location in latitude for Tehran.

As previously stated, 3.69 BTU per square foot of horizontal surface equals one Langley. The total horizontal surface for the pilot plant is 3.1 square feet. Therefore, the total effective solar heat for the pilot plant is as follows:

$$450 \times 3.69 \times 3.1 = 5148 \text{ BTU}$$

f. Determination of the efficiency of the pilot plant: The total heat generated as stated in (d) above is 924 BTU. This quantity divided by the effective solar heat given in (e) above as 5148 is the overall pilot plant efficiency. The result for April 1, 1957 in this plant for the conditions on that day, as aforementioned, is 17.9%.

Losses in the Solar Distiller

Following are our observations of the heat and water losses which take place in our Plant No. 1:

1. Although insulating material under the bottom of the plant will reduce heat loss by conduction, it will not entirely eliminate it.
2. There will be a certain amount of loss through evaporation. Certain unknown parts of the plant will be open to the outside air, leaving room for the vapor to escape.
3. Some rays of light passing through the glass, which is necessary for the vaporization of water, will not only transmit, but will be reflected off the glass back into the atmosphere. The thicker the glass and the higher the presence of minerals in the glass, the lower the transmission rate.
4. Diffuse radiation, present on cloudy days, strikes the glass at all angles, thus creating less transmission than a direct ray.
5. Sash bars, holding the glass in place, are reported to have reduced the energy by as much as 5%, depending on their number.
6. A small loss will be shown through the side walls, and the fresh water channels (collection trays), in that they do not pick up radiation.
7. Some energy, hitting the evaporating pan, will be reflected away—a resultant of the angle of incidence and of the surface and its ability to absorb.
8. Some of the heat in the plant will be picked up by the air by diffusion, and, being warm, will rise. Thus a natural convection is formed. The loss will

depend on the temperature of the water and the glass and will slightly relate to outside factors. Perhaps the best way for offsetting this is by using a vacuum.

9. As too much heat is utilized by deep water, a very thin layer will increase the efficiency, if care is taken to keep the pan level so that the remaining salts do not cause extra reflection back into the atmosphere.

CONCLUSIONS

A. Comparison of Solar Distiller Shape and Dimension

An effort was made to determine the most efficient shape for a simple solar distiller, one that would be practical to construct of the cheapest available material and yet be practical for operation by the local agrarian people.

With the above in mind, a comparison was made, and percentage efficiencies indicated that the cold-frame type of solar distiller is the most practical for isolated areas.

Table No. 3, Comparison of Solar Distiller Shapes and Dimensions, tabulates the various factors which influenced the choice for a particular shape and type of machine. It is noted that the over-all efficiency of Plant No. 3 (air-cooled condenser) is the highest, due considerably to the fact that the frontage is greater in comparison with the other machines.

Plant No. 2 (air-cooled condenser) has the greatest efficiency in glass area. Consequently, this glass arrangement is the best.

Plant No. 1 (air-cooled condenser) is the least efficient. This may be due to the sun's rays passing completely through the machine and being reflected away from the tray. Experiments indicate that placing insulation on the north side of the water-cooling condenser type distiller increased production.

Considering the amount of water remaining in the machine, Plant No. 2 was the most efficient of the air-cooled condenser type machines.

With water-cooling condenser, Plant No. 1 had a higher efficiency than Plant No. 2, due apparently to the narrowness of the machine in proportion to the depth of the tray area.

All of the factors have not been fully evaluated to permit giving definite reasons why a particular shape is more efficient than another, but it is certain that the shape of the machine has a definite bearing on the production and efficiency of a solar distiller.

B. Comparison of Various Arrangements of Solar Distillers

Experiments were made to determine what influence certain accessories or arrangements had on the production of fresh water from raw water sources.

One particular machine was used for all arrangements, thereby eliminating factors of shape or dimension. For these arrangements, Plant No. 1 was used, and Chart No. 3, Installation Chart for the Arrangement of Conditions, is included herein to show the various types of installations.

The comparative expected returns for the various installations of Plant No. 1 for the months of October and November are as follows:

TABLE NO. 2

Relationship of Production to the Temperature of Raw Water, Vapor, and Condensing Surface

Hourly Production Rate

August 7, 1957

Hour	Temp. Raw Water In	Temp. Out-side	Temp. of Pane	Temp. Cool-ing Water	Temp. Vapor	Rate Drops per Min.	Diff. Temp. Cooling Water and Vapor
Plant No. 2							
0800	100°F.	108°F.	39°C.	24°C.	36°C.	10	12° C
0920	110°F.	108°F.	41°C.	27°C.	41°C.	22	14° C
1015	114°F.	108°F.	42°C.	29°C.	43°C.	30	14° C
1140	115°F.	108°F.	38°C.	29°C.	45°C.	38	16° C
1245	117°F.	110°F.	45°C.	29°C.	47°C.	42	18° C
1340	114°F.	110°F.	45°C.	29½°C.	46°C.	40	17° C
1430	113°F.	110°F.	43°C.	29°C.	45°C.	34	16° C
1530	110°F.	106°F.	39°C.	29°C.	41°C.	32	12° C
1610	108°F.	104°F.	36°C.	28°C.	40°C.	16	12° C
1720	93°F.	98°F.	34°C.	26°C.	33°C.	6	7° C
Plant No. 3							
0815	94°F.	90°F.	35°C.	24°C.	38°C.	7	14° C
0930	107°F.	94°F.	39°C.	27°C.	40°C.	18	13° C
1030	108°F.	96°F.	40°C.	29°C.	45°C.	48	16° C
1145	114°F.	102°F.	43°C.	29°C.	48°C.	54	19° C
1245	114°F.	106°F.	42°C.	29°C.	48°C.	68	19° C
1345	114°F.	108°F.	43°C.	29°C.	49°C.	50	20° C
1430	116°F.	108°F.	42°C.	29°C.	48°C.	35	19° C
1530	110°F.	105°F.	41°C.	28°C.	41°C.	40	13° C
1615	110°F.	104°F.	40°C.	27°C.	41°C.	21	14° C
1720	95°F.	94°F.	34°C.	26°C.	34°C.	13	8° C

Note: 200 drops = 9 cc.

TABLE No. 3

Comparison of Distiller Shape and Dimensions

Plants Located in the sun porch

Plant Condition	With Water Cooling				Without Water Cooling (simulating desert Condition)		
Date	Feb. 14 1957		Feb. 15 1957		March 24 1957		
Plants	No. 1	No. 2	No. 1	No. 2	No. 1	No. 2	No. 3
Water in pan c.c.	1650	1650	1065	1240	2276	1675	1531
Water Temp. in pan Degrees F Hours 0730	—	—	—	—	64	62	64
0930	45°	45°	45°	45°	78	87	86
1000	73°	66°	68°	63°	94	97	106
1200	99°	90°	88°	88°	119	124	123
1300	108°	93°	99°	93°	126	126	126
1530	99°	90°	—	—	90	92	93
Tray Area inches sq.	448	361	448	361	448	361	450
Glass Area inches sq.	one Side 485	260	485	260	485	260	550
Frontage inches	26	Clear 15.5	26	15.5	26	15.5	25
Volume of Saturated Air cu.in.	4816	1833	4816	1833	4816	1333	3234
Length of Cooling Coils OD ⅜ inches	176	164	176	164	none	none	none
Distilled Water Returns c.c.	585	410	665	422	100	85	146
B.T.U. per lb. Wat. req'd to evap.	1137	1137	1137	1137	1118	1120	1118
B.T.U. recovered in Distillation	1467	1023	1637	1057	246	213	358
Effective Solar Heat B.T.U.	3272	2655	3405	2674	4886	3965	4919
Percent of Efficiency	44.2	38.4	47.6	39.5	4.9	5.3	7.3
Comparative Efficiencies							
Tray Area Percent %	100	93	100	77.4	100	105.3	145.3
Frontage Length %	100	118	100	108.3	100	143.2	151.6
Glass Area %	100	131	100	128.6	100	159.2	128.8
Cooling Coil %	100	75	100	69.3	—	—	—
Exclusive of Water in "overall" %	100	104.2	100	93.2	100	136	141.9

TABLE No. 3 cont.

Determination of Actual Production of solar distiller

Note — Includes Water heated and evaporated without Water Cooling

Date	March 24 1957			
Plant	No 1	No 2	No 3	
Water in Pan c.c.	2276	1675	1531	
Tray Area inches sq.	448	361	450	
Glass Area inches sq.	485	260	550	
Frontage Length- in.	26	15,5	25	
Saturated Air Volume - cu.in.	4816	1833	3234	
Amount Distilled c.c.	100	85	146	
Water in machine unevaporated (c.c.)	506	374	330	
B.T.U. Recovered in distillation	246	213	358	
B.T.U.- left in machine – Water heated	314	239	205	
Total heat produced in machines — Distilled and Undistilled in B.T.U.	560	452	563	
Effective Solar heat B.T.U.	4886	3965	4919	
Efficiency – %	11,4	11,4	11,4	
Comparative Efficiency in % – including Water heated				
Tray Area	100	100	100	
Frontage Length	100	135	104	
Glass Area	100	184	89	
Over all	100	140	98	

INSTALLATION CHART

Chart No. 3

for Arrangement of Conditions

Installation No.	Clear Pane		Cooling Condenser		Insulation North Side	Coil Condenser		Mirror North Side		Condenser Painted Black North Side	Sun Porch		Outdoors
	S	N	Air	Water		S	N	Inside	Outside		Open	Closed	
1	X	X	X										X
2	X			X						X			X
3	X		X					X					X
4	X			X				X					X
5	X			X		X		X					X
6	X	X		X			X		X				X
7	X	X		X			X						X
8	X	X		X	X		X						X
9	X	X		X			X		X				X
10	X	X		X	X		X						X
11	X	X		X	X		X		X				X
12	X	X	X		X				X				X
13	X	X	X								X		
14	X	X		X			X		X		X		
15	X	X	X									X	

Installation	Water-Cooled	Air-Cooled
#1		20.1 lbs.
#2	25.0 lbs.	
#3		32.1 lbs.
#4	39.3 lbs.	
#5	31.5 lbs.	
#6	47.4 lbs.	
#7	20.7 lbs.	
#8	14.2 lbs.	
#9	20.7 lbs.	
#10	19.8 lbs.	
#11	29.4 lbs.	
#12		22.7 lbs.
#13		13.9 lbs.
#14	12.9 lbs.	
	260.9 lbs.	88.8 lbs.
	(Average 26.1)	(Average 22.2)

(#15 on Chart No. 3 was a later installation during cold weather)

During winter months, considering arrangements 3 and 4 with all conditions relatively similar except the condensing method, the water cooling condensing, instead of air, increases production from 32 to 39 pounds per month or an increase of 22%. Other data available indicates that in summer months when the ambient air temperature is warm, the water-cooling condensing type installation is 300 percent higher in production.

It is evident also that a reflecting surface on the north side of the plant increases production approximately 50 percent.

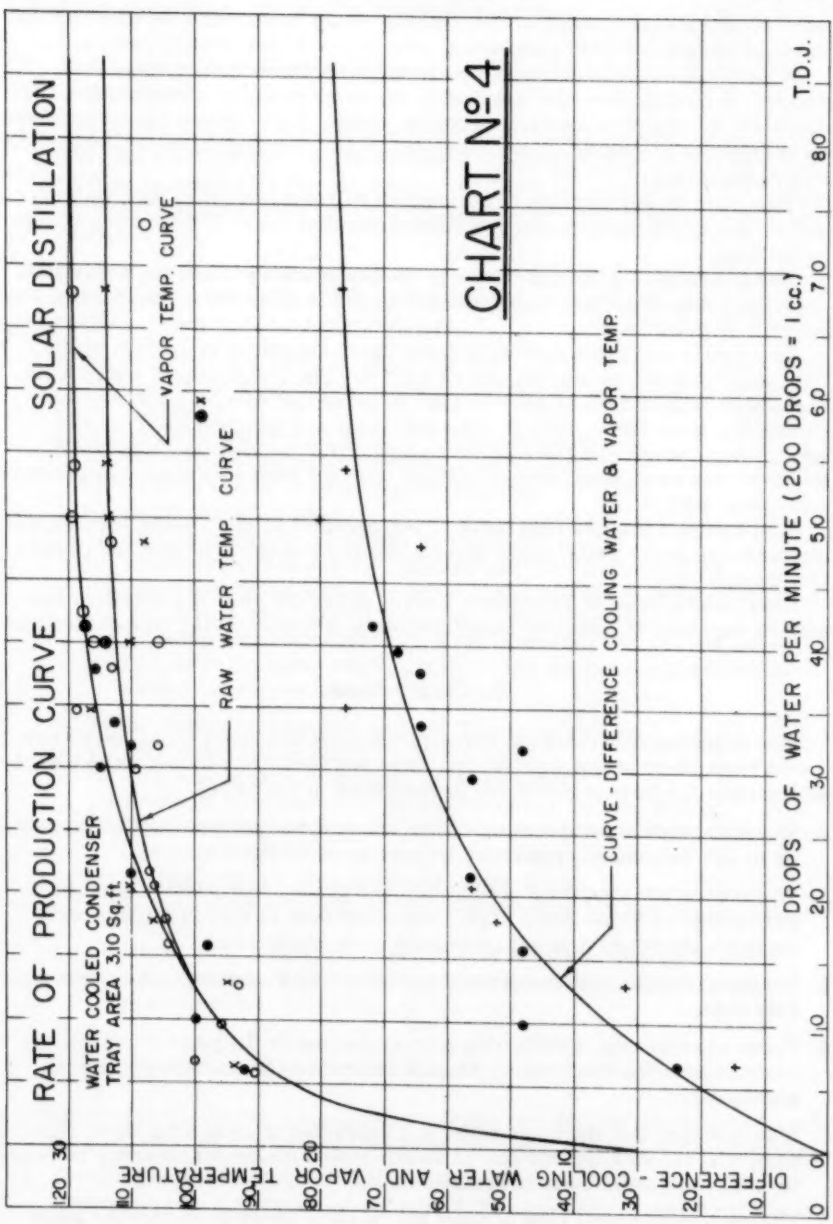
The most satisfactory installation found is Installation #6, comprising clear panes on the north and south sides, water-cooling condenser on the north inside pane, a mirror on the outside of the north pane reflecting the sun's rays into the pan, and the plant located outdoors. The production for this installation is over 100 percent greater than the average of the other installations on which experiments were tried.

C. Relationship of Production to the Temperatures of Raw Water, Vapor, and Condensing Surface

The dew point is the condition in the atmosphere at which vapor in the air is chilled to a point where the vapor condenses to water on an object.

This same phenomenon is prevalent to the solar distiller. However, the dew point in the solar distiller is that instant that the water vapor condenses on the coils of a water-cooled distiller. This does not occur at any specific temperature, but occurs just when the cooling agent drops below the temperature of the saturated air. It was found that the dew point was reached when the temperature of the water vapor was 68° F.; it also occurred at 74° F., and has occurred at 86° F. when the cooling agent was 1/2° to 4° lower than the saturated air.

It was found from experiments that the usual temperatures during the day in arid countries are very little lower than the water vapor to be condensed. Consequently, the production of a solar distiller dependent upon the outside ambient temperature for condensing processes is very low and erratic. Under


SOLAR DISTILLATION
RATE OF PRODUCTION CURVE
CHART Nº 4
WATER COOLED CONDENSER
TRAY AREA 3.10 Sq. ft.
VAPOR TEMP. CURVE
RAW WATER TEMP. CURVE
CURVE - DIFFERENCE COOLING WATER & VAPOR TEMP
DROPS OF WATER PER MINUTE (200 DROPS = 1 cc.)
T.D.J.
DIFFERENCE - COOLING WATER AND VAPOR TEMPERATURE
RAW WATER TEMPERATURE - DEGREES FAHRENHEIT

such conditions, the design of the distiller would necessarily be so large that it would be impractical to construct.

To alleviate this condition, a constant temperature cooling agent was sought. A cooling coil was installed in the solar distiller compartment, the distillate to collect in a trough below the coils. This condensing arrangement increased the production of the distillers in hot climates from 10% efficiency to 35% efficiency.

Table No. 2, Relationship of Production to Temperature of Raw Water, Vapor, and Condensing Surface, tabulates the data taken to determine this relationship.

Results show that the difference in the temperature of cooling water and vapor of 5 degrees Fahrenheit from 14° to 19° in Pilot Plant No. 3 increased production 270 percent.

Production increased gradually from the temperature of 100° F. of the raw water to a maximum obtained of 116° F. The production of water from raw water temperature of 100° to 116° F. increased 400 percent over the production from 76° to 100° F. The dew point was reached when the raw water temperature was 76° and the cooling water temperature was 75° F. However, the production of the distillate was low when raw water temperature was below 100° F.

It is believed that the difference of temperature of the cooling agent to the temperature of the water vapor determines the amount of production of the distillers.

Chart No. 4, Rate of Production Curve, shows the rate of production that may be expected at indicated temperatures in a water-cooled type of distiller.

D. Observations

The following observations, recommendations and conclusions have been drawn from study of the distiller reaction; and they are believed to be useful information for further profitable development in the future.

1. The difference in temperature of the saline water and the condensing pane, up to 25° Fahrenheit, produced the maximum condensate.
2. To produce the maximum condensate, the transparent surface must be perpendicular to the sun's rays. The north side is mainly a condensing surface—therefore it is not necessary to be transparent.
3. Having the glass pane inside the foundation takes advantage of any available rain water.
4. From experiments, a reflecting mirror located in the plant to reflect the sun's rays to the evaporating surface increases the efficiency approximately 100%.
5. It is assumed that the north pane, if constructed of reflecting sheet aluminum, will act as a mirror and as an air-cooled condenser, thereby increasing efficiency over those presented herein.
6. The most economical type of plant for isolated areas is an air- or water-cooled condenser, cold-frame type.
7. For practical structural requirements, double strength glass or plate glass should be used, and the evaporating pan should be painted a dull black to absorb radiation.

8. Waterproofing to prevent leaking lessens the percent of heat loss.

9. A partial vacuum would be effective in reducing the heat loss through air, and would lower the evaporating temperature, thus increasing the plant efficiency.

10. A thin film of water is most effective. Just enough to cover the pan so it will heat more rapidly, but sufficient so that no dry salts are remaining. With an excessive amount of water, the efficiency rate is lowered, because the available heat for evaporation is used to heat water not evaporated.

11. Of the rays reflected back from the pan bottom, some will reflect off the glass onto the water and back again an infinite number of times. This creates a very small measure of loss.

12. From observations taken to March 28, 1957, the following requirements were considered to be the most important factors for creating a complete solar machine:
    a. Air-Cooled Condensing Surface Type Installation
        (1) Low sides—sufficient slope for gravity flow.
        (2) Frontage 150% of depth (from records).
        (3) Reflecting back pane.
        (4) To avoid shadow, use glass on three sides.
        (5) Frontage pane should be perpendicular to the sun at noon.
        (6) The water depth should be equal to the amount of daily evaporation; this varies with time of year and of temperature.
        (7) Location—unobstructed from wind currents and total sun rays.
        (8) Ridges in the glass should incline down the pane, not crosswise—easier for movement of water.

    b. Water-Cooled Condensing Surface Type Installation
        (1) Frontage 150% of depth (from records).
        (2) To avoid shadow, use glass on three sides.
        (3) Frontage pane should be perpendicular to the sun at noon.
        (4) The water depth should be equal to the amount of daily evaporation; this varies with time of year and of temperature.
        (5) Location in sun area, protected from wind.
        (6) Glass ridges immaterial (this method does not use the pane for the rivulets).
        (7) Relation of cooling coil area to tray area variable with outside temperature; to be controlled by amount and temperature of condensing water.

E. Illustration of the Practical Application of Solar Distillation

1. Assume that each person requires 1 gallon per day for drinking and cooking purposes, or a family of five requires five gallons of water per day.

2. The total returns for April 1957 for Pilot Plant No. 1 are: 4124 cubic centimeters or 9.1 pounds of water at 8.3% efficiency equals 3 pounds of water collected per square foot of tray area.

3. The evaporating pan area required (for a month under these conditions—April records) is:

5 gallons per day x 30 days or 150 gallons per month. 150 gallons required x 7.5 pounds per gallons of water equals 1125 pounds of water required for the month. 1125 divided by 3 pounds per square foot area equals 375 square feet of tray area required for a family of five at one gallon per person per day.

4. Records indicate a maximum average efficiency of 20% for summer months or 2.5 times the production for April (8.3%).

5. Thus, expected returns for summer: 2.5 x 5 gallons per day x 30 days or 375 gallons per month or 2.5 gallons per capita per day.

6. If this plant were built with reflected sheet aluminum (Alcoa), on the north side (to act as a mirror), the expected returns would be approximately double (obtained from short period records), or 750 gallons per month or 5 gallons per capita per day. This is considered an adequate amount for drinking and cooking in remote isolated areas.

7. It may be possible and desirable to blend this water production with other water to increase the total potable supply and reduce the total cost per 1000 gallons. However, a filter would be necessary to purify the blended water. This is desirable to increase the mineral content which will make the water more palatable. Distilled water is flat and tasteless.

8. Cost of Plant

A preliminary cost estimate of the plant described in 1-6 above, for Iran, is as follows:

Excavation	600 Rials
Concrete, 50 x 8.5 x 1/3'	11,000
Waterproofing	1,000
Gravel fill	750
Steel work	18,900
Glass, 12 sq.m. @ 250	3,000
Sheet metal ends, 4 @ 200	800
Misc. Expenses	3,000
Forming	3,300
Aluminum (Alcoa) 200 sq. ft. @ 25¢	3,750
Freight	1,000
	47,100 Rials or $628.00

If brick walls were used, the cost would be decreased but, likewise, production would be decreased.

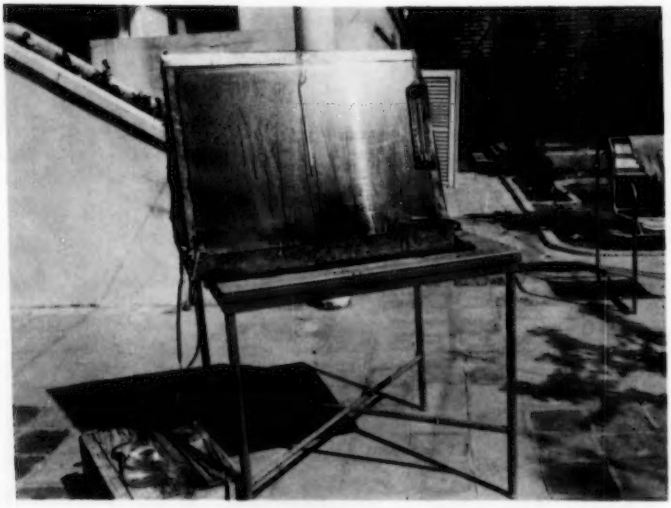
The direct operating cost is not considered as this will be a function of the family—one of the chores.

The annual financial cost is as follows:

Interest 5% on $628.00	$31.40
Depreciation, 5%	31.40
Upkeep and repairs	10.00
Total estimated annual cost	$72.80

The estimated water costs are as follows:

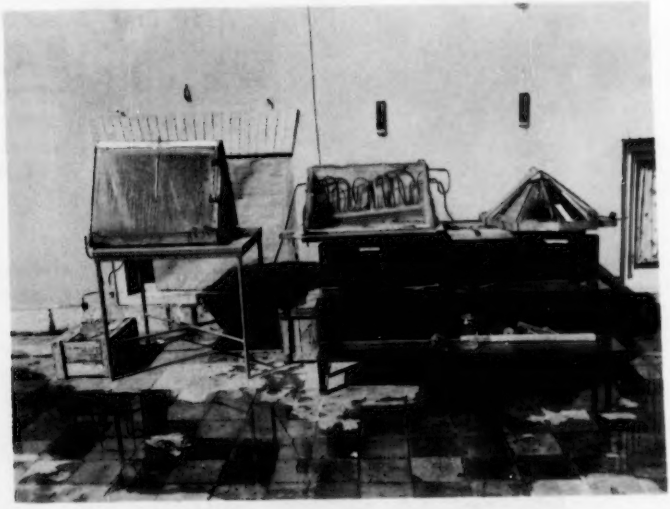
750 gallons per month or 9000 gallons per year.
$72.80 divided by 9000—$.008 per gallon.



Plant No. 1

Plant No. 2

Plant No. 3 in foreground



Pilot Plants
No. 1 No. 3 No. 2

As mentioned on page 2, some people are currently paying as much as $1.25 for five gallons of drinking water.

BIBLIOGRAPHY

1. Demineralization of Saline Water with Solar Energy Research and Development Progress Report No. 4 Saline Water Conversion Program by George O. G. Lof, under contract No. 14-01-001-55 U. S. Department of the Interior, Washington, D. C., August 1954.
2. Elementary Practical Physics Newton Black and Harvey Davis, Macmillan Co., New York, N. Y., 1953.
3. Encyclopedia Americana "Solar Radiation" under "M - Meteorology"
4. Fresh Water from the Ocean Cecil B. Ellis and members of the staff of Nuclear Development Association, Inc., White Plains, N. Y. The Ronald Press Co., New York - 1954.
5. Handbook of Applied Hydraulics Section 2, Hydrology, by C. S. Jarvis 1942 edition.
6. Hydrology Handbook American Society of Civil Engineers, New York, N. Y., 1949.
7. Mechanical Engineers' Handbook, page 1262, Lionel S. Marks, 1941 Edition.
8. Notes on Economic Consideration of Fresh Water Production by Solar Distillers (Preliminary Copy) (Hervol-Leet) Massachusetts Institute of Technology Solar Energy Research Project No. 2743, 1954-55.
9. Latin American Reports (Water Distillation Plant - Aruba) Latin American Reports 1956, International Trade Mart, Inc., New Orleans, La.
10. Richards' Topical Encyclopedia Goolier Society, Inc., New York, N. Y., 1955.
11. Saline Water Conversion Annual Report of the Secretary of the Interior for 1955, U. S. Department of the Interior, Washington, D. C., January 1957.
12. Saline Water Conversion Report for 1956 U. S. Department of the Interior, Washington, D. C., January 1957.
13. Sea Water Conversion Program A Progress Report for the State of California Legislature University of California, Institute of Engineering Research, Berkeley, California, March 1, 1955.
14. Seasonal Variations in the Atmospheric Heat Balance A Doctoral Thesis for Massachusetts Institute of Technology, by J. Gabites.
15. Solar Radiation as a Power Source C. G. Abbott, Smithsonian Institution, Washington, D. C., 1943.
16. Use of Solar Energy for Heating Water T. A. Brooks, Smithsonian Institution, Washington, D. C., 1939.
17. Utilizing Heat from the Sun C. G. Abbott, Smithsonian Institution, Washington, D. C., 1939.

Appendix A

TABLE NO. 4

Two-Way Temperature Conversion Table

EQUIVALENT VALUES OF DEGREES, FAHRENHEIT AND CENTIGRADE

C		F	C		F	C		F	C		F
-40.0	-40	-40	-2.78	27	80.6	11.7	53	127.4	28.3	83	181.4
-34.4	-30	-22	-2.22	28	82.4	12.2	54	129.2	28.9	84	183.2
-28.9	-20	- 4	-1.67	29	84.2	12.8	55	131.0	29.4	85	185.0
-23.3	-10	14	-1.11	30	86.0	13.3	56	132.8	30.0	86	186.8
-17.8	0	32	-0.56	31	87.8	13.9	57	134.6	30.6	87	188.6
-17.2	1	33.8	0	32	89.0	14.4	58	136.4	31.1	88	190.4
-16.7	2	35.6	0.56	33	91.4	15.0	59	138.2	31.7	89	192.2
-16.1	3	37.4	1.11	34	93.2	15.6	60	140.0	32.2	90	194.0
-15.6	4	39.2	1.67	35	95.0	16.1	61	141.8	32.8	91	195.8
-15.0	5	41.0	2.22	36	96.8	16.7	62	143.6	33.3	92	197.6
-14.4	6	42.8	2.78	37	98.6	17.2	63	145.3	33.9	93	199.4
-13.9	7	44.6	3.33	38	100.4	17.8	64	147.2	34.4	94	201.2
-13.3	8	46.4	3.89	39	102.2	18.3	65	149.0	35.0	95	203.0
-12.8	9	48.2				18.9	66	150.8	35.6	96	204.8
-12.2	10	50.0				19.4	67	152.6	36.1	97	206.6
-11.7	11	51.8				20.0	68	154.4	36.7	98	208.4
-11.1	12	53.6				20.6	69	156.2	37.2	99	210.2
-10.6	13	55.4				21.1	70	158.0	37.8	100	212.0
-10.0	14	57.2	4.44	40	104.0	21.7	71	159.8	43	110	230
- 9.44	15	59.0	5.00	41	105.8	22.2	72	161.6	49	120	248
- 8.89	16	60.8	5.58	42	107.6	22.8	73	163.4	54	130	266
- 8.33	17	62.6	6.11	43	109.4	23.3	74	165.2	60	140	284
- 7.78	18	64.4	6.67	44	111.2	23.9	75	167.0	66	150	302
- 7.22	19	66.2	7.22	45	113.0	24.4	76	168.8	71	160	320
- 6.67	20	68.0	7.78	46	114.8	25.0	77	170.6	77	170	338
- 6.11	21	69.8	8.33	47	116.6	25.6	78	172.4	82	180	356
- 5.56	22	71.6	8.89	48	118.4	26.4	79	174.2	88	190	374
- 5.00	23	73.4	9.44	49	120.2	26.7	80	176.0	93	200	392
- 4.44	24	75.2	10.0	50	122.0	27.2	81	177.8	99	210	410
- 3.89	25	77.0	10.6	51	123.8	27.8	82	179.6	100	212	413
- 3.33	26	78.8	11.1	52	125.6						

Paper 1705 July, 1958 SA 4

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

SANITARY ENGINEERS: THE NEED AND THE SECURING

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(Proc. Paper 1705)

ABSTRACT

Growing population and accelerating technology are prime factors in complicating environmental problems. Therefore, more, better and differently qualified sanitary engineers are required. They should be attracted by a more adequate public relations campaign, and should be financially supported at the academic training levels especially in the undergraduate years.

Part 1 - Manpower Aspects

The sanitary engineer's horizon has expanded rapidly in recent years. From his early days in this century as a civil engineer with primary concern for the construction and maintenance of waterworks, he has been called on increasingly to meet and solve a comprehensive array of environmental problems, either potential or actual-mass housing, vector control, water resources, air pollution, and radiation. He is often called on to handle both ends of problems-to establish and elaborate the engineering aspects of preventive medical practices and to set up and maintain control programs for the alleviation or elimination of problems which have already become public health hazards. Faith in the engineer's ability to control the environment is frequently tested by the demand that the sanitary engineer contain certain community health hazards in advance of the medical discovery of adequate means for controlling such hazards.

Note: Discussion open until December 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1705 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 4, July, 1958.

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Anyone doubting the strategic role of the sanitary engineer in our society is reminded of the two major factors responsible for this Nation's continued growth and standard of living — increase in population and technological advance. Sixty million more people will be added to our Nation in the next 18 years.(1) The acceleration in new products, processes, and techniques is dramatically indicated by the estimate that, on the basis of the average rate of technological progress since 1850, this country should be able to produce in the year 2000 as much in one seven-hour day as is now produced in one 40-hour week.(2)

Both factors — people and technology — result in rapid environmental changes in our ways of living, some of which are fraught with dangers both individual and collective to our health.

How Many Sanitary Engineers are There?

The exact number of practicing sanitary engineers is not known, though two approximate estimates are available. Still in process of analysis is a comprehensive questionnaire survey undertaken by the National Science Foundation and the U.S. Public Health Service on behalf of the National Register of Scientific and Technical Personnel. Tentative and preliminary organization of the data suggest that 5,500-6,500 represents a reasonable figure for the total sanitary engineering population. Also the number of sanitary engineers graduating with first degrees from engineering schools in the United States has in recent years fluctuated around one per cent of the total yearly graduating class of all engnieering groups,(3) as is indicated below:

Year	Engineers*	Sanitary Engineers	Per Cent Sanitary Engineers of All
1956	28,836	341	1.18
1955	25,243	286	1.13
1954	24,427	293	1.19
1953	25,960	323	1.21
1952	31,873	330	1.14
1951	43,624	403	0.9
1950	53,217	439	0.8

*ECPD totals

In view of the job opportunities available at various governmental levels and estimated attrition rates, it is suggested that approximately three times the number of currently graduating sanitary engineers are needed annually in the next ten years to make up the deficit.

Sanitary Engineering Graduates, 1900-1956

Various records were examined to note trends, if any, in the academic production of sanitary engineers. The accompanying table lists the number of undergraduate and graduate degrees awarded in this field since the turn of the century.

A first perusal of these data may appear to give the lie to the assumed shortage of sanitary engineers. Inspection of the figures does, indeed, disclose a healthy trend: that of all the bachelor degrees awarded since 1900

in this field, almost half were given in the last ten years, 1946-56; that of all the Master's degrees earned in this field since 1900, almost two-thirds were earned in the last ten years; and that of all the Doctor's degrees awarded in this field, almost three-fourths were presented in the past decade. The reaction could well be: "the sanitary engineers are doing fine! They never had it so good!"

Manpower Questions Still to be Answered

Further consideration of this table raises some strategic questions which must be answered before any satisfaction is to be derived from such a pronounced acceleration in the production of sanitary engineers. Do all those graduating with the first degree actually enter upon their first job in this field? What is the attrition rate? For example, information gathered by the National Scientific Register suggests that approximately 20 per cent of living graduates of engineering schools were engaged in non-engineering work in 1953.(4) In a group as small as the sanitary engineers, such a proportion of drop-outs could be significant and damaging.

Still other questions must be answered. How many positions in sanitary engineering are currently filled by individuals trained in other engineering areas? Are they so filled because of the lack of properly qualified sanitary engineers? The questions remains-what is the NEED for sanitary engineers? Is the current yield of degree-awarded sanitary engineers sufficient to assist in establishing, maintaining, and extending the Nation's health?

It is the authors' conviction that these questions must be answered if our country's health standards are to be maintained, and it is believed that agencies both in and out of the Federal Government are interesting themselves in this particular aspect of engineering manpower assessment.

Forecasting Need for Sanitary Engineers

The historic role of the sanitary engineer in the past 50 years in overcoming the communicable diseases, his control and guardianship of the Nation's water resources and water uses, the pros and cons of this specialty as a professional group, explanation for the small number, the need for a curriculum in this field longer than the conventional four-year training period-these and related subjects are left for other presentations.

For purposes of this paper, it may be initially desirable to venture into the forecasting business by way of estimating future new or expanded needs for sanitary engineering services. Despite the lack of a major, statistically designed national survey, or even a relatively small sample survey, certain trends may be noted. By way of illustration, two new areas and one traditional concern will be described, each in different ways but all within the sanitary engineer's horizon.

Air Pollution Developments

The increasing problems relating to atmospheric pollution may be considered more representative than typical of the impact of technological advance on the environment. (It must also be noted that, too often, the sanitary engineer is brought in on problems as they develop and rarely is provided the luxury of setting up adequate and necessary safeguards in advance of the problem stage.)

"The field of air pollution is concerned with the sources, extent, physical and chemical behavior, and effects of contaminants introduced into the outdoors atmosphere by economic and community activities, excluding the agents of communicable disease, and the procedures for the control or prevention of such contaminants."(5)

This comprehensive definition is directed squarely at the sanitary engineer. A recent study (6) indicates that approximately 10,000 communities in this country have air pollution problems. Well over 100 have formalized some means of combatting this problem, ranging in size and comprehensiveness from the mayor's citizens' committee in a small city of 17,000 souls to the multimillion dollar operation in Los Angeles. A tabulation of the number of State and local agencies engaged in air pollution control activities shows the total of 114 in September of 1955 more than doubling by September of 1956. The most recent Directory of Governmental Air Pollution Agencies(7) lists programs in 47 States and Territories and the District of Columbia. Including those operating at the State, county, and municipal levels, 72 are full-time programs, 69 are part-time, and 140 are set up to at least answer specific compliants or requests for information.

Nine States and one Territory have some kind of air pollution legislation. These include California, Oregon, Washington, Kentucky, Massachusetts, New York, New Jersey, Florida, Colorado, and Hawaii. So far, in calendar year 1957, some 82 bills have been introduced in various State legislatures.

The Air Pollution Control Association has grown phenomenally in recent years, reflecting professional concern. One regional section of this group held a meeting recently with members attending in numbers which represented the national organization's total membership as recently as five years ago.

Both the Federal Government and institutions of higher education have made increasing provision for research in this field. Public Law 159 (1955) was enacted by the Congress to aid and support technical research to devise and develop methods of abating air pollution. Initially, a maximum $5,000,000 yearly expenditure for five years was legislated.(8)

Currently, 68 colleges, universities, and research institutes are engaged in air pollution research, as compared with 45 in 1955 and 37 in 1953. Research topics include control of impurities in automobile exhausts, development of instruments to sample and analyze various atmospheres, measurement and analysis of air impurities, and the influence of smog on plants and health.(9)

As of July 1, 1957, some 37 research grants in air pollution, involving approximately $874,000 are sponsored by Federal funds. A number of outstanding private research institutions are increasingly involved in this field.

Obviously, engineers need more training in this field than they are now getting. "We need education of young engineers at college level in order to provide them with basic information on techniques of air pollution control and performance of equipment which can be applied to process effluents."(10)

Other disciplines and engineering specialists are increasingly helpful in this field. The biologist and chemist can be found in many laboratories and communities working on facets of air pollution. The chemical and mechanical engineer also have a strategic role in combatting and controlling atmospheric contamination.

The development of new professional opportunities in this field in industry, Government, and universities; the addition of research fellowships in this

field, especially by Public Health Service; and the availability of some new textbooks in air pollution[11] have stimulated some of the leading academic institutions to either develop new or expand graduate curricula in atmospheric pollution. Currently, the following schools have such curricula-Harvard, Rutgers, California, Cincinnati, Florida, Michigan, North Carolina, and Pittsburgh-and more may be expected.

Research in air pollution is slow, painstaking, and complex. Despite the current lack of comprehensive control techniques, enough is known to cause some communities to set up positions requiring competence in the health aspects of air pollution. As a result, certain of the "trade journals" have come up with an increasing number of "men wanted" ads, asking for sanitary engineers with training in air pollution control.[12]

In brief, it would appear that as problems of air pollution become more serious; as the public becomes increasingly roused to action; as research discloses new and more effective means of controlling atmospheric pollution-it may be expected that the more than 100 communities with air pollution abatement committes and the estimated 10,000 population centers with air pollution problems will all call upon local authorities with increasing persistence to acquire professional and technical staff necessary to handle these problems. The need for qualified sanitary engineers in this field can be expected to grow steadily.

The Comming Nuclear Technology

The field of nuclear energy could be elaborated upon in similar fashion but can be capsuled at this point. The estimate by the Atomic Energy Commission that by 1980 as much as one-third of the Nation's total kilowatt production could come from nuclear reactor plants;[13] the activity of the Sanitary Engineering unit of the AEC; the more than $1,500,000 of current research with AEC money in the problems of radiation hazards, health, and safety; the phenomenal growth in the use of radioisotopes in industry; the potential of the dispension of radioactive wastes into the streams and atmosphere of the Nation-these situations, too, point to an expanding area of need for the services of the adequately trained sanitary engineer.

Water Resources Development

In comparison with the relatively new problems involving atmospheric pollution and radiological health hazards, the fields of water resources and water quality management may appear as well-worn and traditional, with most of the problems already solved. Of course, this is hardly the case. In fact, the problems of finding, using, and conserving water to meet the needs of 60,000,000 more additions to the national population in the next 18 years will greatly challenge the engineering, operational, research, and administrative skills of the sanitary engineer.

The following briefly identified problems are representative of the range of activities that require more and continually better qualified sanitary engineers, and pose operational and research opportunities surprisingly fresh for a "traditional" area: upstream retention-precipitation is adequate but fluctuations result in "feast or famine;" upstream movement — potential use of atomic power in less populated areas; evaporation control; weather management — rain-making possibilities; watershed control; control of new types of pollution — advance design of equipment and chemical treatment; reuse

of water, especially for industry; desaltation potentials; and the recharge of ground water aquifers.

CONCLUSION

Examination of sanitary engineering manpower statistics leaves much to be desired in order to properly determine the size and nature of the professional corps needed to adequately guard the public health. The growth of population and advance of technology are both so overwhelming as to clearly suggest expanded need for checking, identifying, analyzing, and controlling the many resulting environmental changes which have actual or potential danger for the national well-being. Part II will deal with methods of creating greater interest in promising young students to consider sanitary engineering as a career choice.

UNDERGRADUATE AND GRADUATE DEGREES AWARDED IN SANITARY ENGINEERING IN THE UNITED STATES, 1900-1956

Year	Bachelor's[a]	Masters'[b]	Dr.'s[c]	Year	Bachelor's	Master's	Dr.'s
1900-34	1,265	207	5	1946	28	60	2
1935	86	35	1	1947	112	143	1
1936	88	18	1	1948	160	163	4
1937	75	51	2	1949	247	136	4
1938	82	42	1	1950	287	148	4
1939	103	51	2	1951	244	152	7
1940	116	70	2	1952	216	105(22)[d]	9
1941	98	68	4	1953	216	102(20)	5(3)[d]
1942	93	45	2	1954	164	120(25)	9
1943	165	24	4	1955	141	134(34)	11(2)
1944	65	32	-	1956	208	124(31)	9(1)
1945	40	33	-				
				Total	4,299	2,064(421)	89(12)[e]

a-No distinction is made between nationals and non-nationals awarded the degree. See footnotes "d" and "e".
b-Of the total of 1,478, 1900-1951 inclusive, 289 were awarded to nationals of other countries.
c-The first doctorate degree in sanitary engineering on record was awarded by Harvard University in 1928.
d-Figures in parentheses represent nationals of other countries.
e-Of the total of 46, 1928-1951 inclusive, 6 were to nationals of other countries.

REFERENCES

1. U. S. News and World Report, August 9, 1957, pp. 46-54. Based on studies by U.S. Bureau of Census, regional planning groups, and industrial surveys.
2. Barish, Norman N. (ed.) Engineering Enrollment in the United States. New York, New York University Press, 1957. P.6; see also reports of the U.S. Patent Office; and The Chemical Industry Facts Book, third edition, Manufacturing Chemists' Association, Inc., 1957 as illustrations.
3. Comparisons are readily made by reference to yearly engineering totals as prepared in various issues of the Journal of Engineering Education and yearly sanitary engineering totals as found in issues of Public Health Reports (U. S. P. H. S.).
4. Blank, David M. and Stigler, George J. The Demand and Supply of Scientific Personnel, N.Y., National Bureau of Economic Research, Inc. 1957, pp. 70-71.
5. From Report of Ad Hoc Committee to Chairman, Sanitary Engineering-Occupational Health Study Section, National Institutes of Health, PHS, DHEW, November 30, 1955. Page 3.
6. Arthur C. Stern, "Air Pollution Control Administration Needs and Patterns," First International Congress on Air Pollution. American Society of Mechanical Engineers, New York. 1955.
7. Published by Air Pollution Control Association in cooperation with the U. S. P. H. S.
8. See Public Law 159, 84th Congress, Chapter 360, First Session, S.928.
9. Industrial Wastes, volume 1, p. 71 (May-June, 1957)
10. Leslie Silverman, "Engineering Research and Development in Air Pollution Control", Chapter 2 in Problems and Control of Air Pollution, F.S. Mallette (editor). Page 44.
11. Arthur C. Stern, "Present Status of Atmospheric Pollution in the United States, "Journal, Amer. Pub. Health Assoc., 47:78-87 (January, 1957).
12. See Journal, Amer. Pub. Health Assoc., September and October, 1956, and March, 1956; Air Pollution Control Association News, September and October issues, 1956; and Civil Engineering, December, 1956, as examples.
13. Paper by W. K. Davis and L. H. Roddis, Jr. (Atomic Energy Commission), "The Latest Prospects for Economic Nuclear Power," presented at Fifth Atomic Energy in Industry Conference of National Industrial Conference Board, Philadelphia, March 14, 1956. Pages 7-8.

Part II - Getting More Students

The need for more sanitary engineers is based on two assumptions: (1) that a shortage exists; for example, a number of local Public Health Departments indicate vacancies; and (2) the opening of new fields of potential or actual public hazard, such as radiological and atmospheric.

There has been a considerable drop-off in the award of undergraduate and graduate degrees in sanitary engineering from the high point reached in 1950.

Why Students Avoid the Profession

Search for explanation of this slump led to the students themselves. Students in scattered institutions and schools were either formally or casually asked to explain their disfavor of sanitary engineering as a career possibility. The answers were uniform. As reported by Boyce, (1) some of his students felt that very little information was available on the importance, requirements, and potential of the field. As a result, they considered the field as lacking in glamour, lacking sufficient financial remuneration, and lacking in problems requiring solution.

This paucity of information as a stumbling block to entrance into sanitary engineering was noted in a United States Public Health Service study in 1954, when Rosenstock and Miller (2) recommended that the profession "present the undergraduate student with a more adequate picture of sanitary engineering work, its specialties, its possibilities, and its future."

It cannot be denied that the public relations program of this branch of engineering has been either non-existent or poorly carried out. Sanitary engineering does, indeed, lack glamour when one considers the drum-beating given to stirring the imagination of science-minded youngsters with data on jet propulsion, nuclear energy, and guided missiles. Compared with the attention given to professional activities literally "out of this world," the complications of the water, air, food, and other environmental aspects of this world surely pale into insignificance! Certainly the professional engineering aspects of sanitary engineering have not been adequately pointed out, when students suggest this specialization to be primarily in the medical field.

Changes in Attitude Noted

The situation is far from hopeless, however. A few studies of student attitudes toward careers suggest that sanitary engineers have increasingly good opportunity to be heard. One student opinion survey discloses a shift from primary concern with salary to one involving professional experiences. When asked what he really wanted from a career, the graduating senior listed "opportunity to learn" and "interesting work" ahead of "good salary." (3) One explanation of this rising concern for professional experience is probably the realization that any career these days will provide, at a minimum, the necessities of life. Beyond these, it is increasingly recognized that the desired remuneration will come from the matching of an adequate academic background with pertinent, comprehensive, on-the-job exposures.

If this reasoning is valid, the sanitary engineer has a real story to tell, and young people want to hear it. In contrast to the criticisms provided,

some of Professor Boyce's students [4] furnished constructive suggestions for stimulating interest in the field. These included: finding a new name for the work; giving scholarships on the basis of high school sanitation projects; and contacting students earlier and supplying them with more information about sanitary engineering. It would be of interest to examine the other side of the coin and query students who did select the sanitary engineering option and who received their first degree in this field. What influenced their decisions: a favorite faculty personality? — proximity of the school to home? — financial assistance? — the availability of a related summer job? — the closest field to a thwarted medical career? — real desire to serve the community? Such students could conceivably furnish excellent background, information, and points of view for use on their foot-dragging, uninterested peers.

The Story That Must Be Told

Obviously, the sanitary engineering story must be told in greater detail, more interestingly, and to more students. With the discovery of science interest in as low as the elementary grades, it is necessary to begin with at least the high school student. Certain major themes rate priority. These are: the professional engineering aspects of the field; the link of engineering and health; the many problems still to be solved and the emergence of ever-new problems; and the variety of talents that can find pace in the field. The profession must be dramatized in all its facets — the human relations and health aspects, the strategic role of the sanitary engineer in solving the environmental problems arising from our advancing technology, the opportunities for travel, and the diversity of experiences to encounter.

It is worth emphasizing that the present concentration of recruitment effort in the final half year or just after graduation from college is timed too late to attract the number of students into sanitary engineering sufficient to meet even present minimum needs for such people.

Every Engineer A Recruiter

The individual and coordinated effort of every sanitary engineer and professional organization is required to elaborate on these features of the profession. Every sanitary engineer must consider himself a recruiter and must not overlook any opportunity to describe and discuss his field, whether it be in the course of professional duties or by participation in civic activities.

In order to assist in this promotional campaign, professional groups with and interest in sanitary engineering should develop material geared and oriented to the elaboration of the various aspects of sanitary engineering. Films, brochures, graphic displays and exhibits, professional papers, and other kinds of materials must be developed and a variety of communications media employed. Some should be prepared for those already in the profession in order to solicit their assistance and stimulate greater interest. Other material should be specifically aimed at and distributed to high school students and undergraduate college students through science teachers, vocational guidance groups, and faculty advisors at all levels.

Films on Sanitary Engineering

The U.S. Public Health Service has undertaken a number of projects which are described here briefly to illustrate and suggest the media and materials

possible in bringing attention to this field. The production of a 15-minute, sound, color movie on sanitary engineering is under way. The film utilizes incentive-motivation techniques, and should prove interesting to students in the upper high school grades and freshmen and sophomores at the collegiate level. A brochure of questions and answers will accompany each reel so that the film may be shown within the larger framework of a serious discussion of career opportunities in sanitary engineering. Similar productions, either of the movie "short" type or slide, strip films, could well be a project for the communications department of a college or university. Cooperation of Liberal Arts and Engineering department faculties with senior students in such a "learning-how-to-make-a-movie" project could produce very usuable materials in local or regional situations. The authors know of at least two outstanding universities that carry on just such activities.

Vocational Guidance Material

The Public Health Service is also issuing a vocational guidance brochure on sanitary engineering for distribution to thousands of high school vocational guidance specialists and science teachers, college career counselors, engineering faculties, sanitary engineers, and other interested individuals and agencies. This brochure is attractive in format and is written in simple and direct language. Representative headings in the pamphlet include Matching Interest and Ability, the Future Needs More Sanitary Engineers, A Career For Many Talents, and High School and Collegiate Courses Most Useful for Sanitary Engineering. Here, again, there is no monopoly. An interested and aggressive sanitary engineer would be welcome in the vocational guidance office of the local public school system for the purpose of working up a brief mimeographed statement on sanitary engineering which would have the "local touch." There is also need for better teamwork between those authorities in high schools and in colleges who are responsible for vocational guidance.

High School Science Material

A series of demonstrations and experiments is being developed under contract from the Public Health Service by a committee of carefully selected, competent, high school science teachers. The committee is translating the research activities of the Service's Robert A. Taft Sanitary Engineering Center into a set of experiments having the potential of reproducibility in the high school physics, chemistry, and biology classroom. In this manner, the students will be given some practical and interesting illustrations of many scientific principles, and, at the same time, will be introduced to many of the activities of the operating sanitary engineer. This, too, is a project that has many potential counterparts throughout the country. The aggressive sanitary engineer, whatever his professional role, can aid in bringing university engineering faculty into closer working relationship with public school teachers in the particular area. To illustrate, one Public Health Service Regional Engineer's office arranged to have some science teachers in the metropolitan area's junior colleges man the air sampling network stations on a part-time basis. In turn, these teachers brought some air pollution filters and other instruments of detection, measurement, and analysis directly into their classrooms for demonstration purposes. Interest was naturally high.

Summer Student Employment

Greater interest in and knowledge of sanitary engineering can be gained by the student through summer employment during undergraduate years. On-the-job exposure of 60-90 days during the summer months could aid significantly in inducing promising students to consider sanitary engineering as a desirable career. Those employing summer students should plan their program carefully so that the student gets a clear understanding of a variety of sanitary engineering functions, operations, and responsibilities.

During the summer of 1957, the Public Health Service inaugurated a program of summer employment specifically for engineer undergraduates (some sanitary engineering "majors," others not) in its Commissioned Officers Student Training Extern Program (COSTEP). Fourteen young collegiate sanitary engineering "majors" from institutions throughout the country were selected for summer assignment to various Public Health Service installations and programs. Preliminary evaluation indicates this to have been a most instructive and salutary experience for both the Service and the young men selected. Many universities make such arrangements with local, county, and State departments of health, but many more could, that do not. Sanitary engineers everywhere could take the initiative in providing such temporary employment on their staffs and, through such students, can pass on information about the profession in the most effective manner possible — namely, student talk about real, remunerative, on-the-job experience.

The high school student need not be overlooked in such a summer activity. The Newark, New Jersey, water works hired some 30-40 high school students to participate in maintenance work during the conventional vacation period. The authors understand that the response, experience, and exposure were most gratifying to all involved.

Many communities are in position to attract and interest young people to careers. The cooperation of the local health department with the public schools via the sanitary engineer could provide many summertime exposures to the field for promising students. They could be easily identified and selected on the basis of such factors as classroom grades, intelligence quotient, interest in science, teacher recommendation, and parental approval.

Advancing the Profession in the Role of Active Citizen

Many sanitary engineers work closely with a variety of key public officials. Such professional and personal relationships can be most ethically taken advantage of in terms of setting up opportunities to describe and discuss the profession Membership in a local chapter of an engineering association or even in a social service group (Rotary, Kiwanis, Lions, etc.) could provide the sanitary engineer the chance to involve his chapter or represent his chapter in the local high school "career week" program or the nearby college freshmen "orientation week." The stage is then set for the talk on sanitary engineering to audiences willing to listen.

Colleges have junior engineers clubs before which the sanitary engineer could seek audience. High schools have extra-curricular science clubs and carry on elaborate science talent competition. The sanitary engineer would do well to seek participation as an adviser and/or judge along with other prominent community-minded citizens. He would be most welcome.

In some cities, active adult education councils or denominational groups plan series of public service lectures covering a variety of international, national, and local problems. The theme might be "Our Growing Community" or "Living Longer in Our City." The human interest motif is already "built in" in any topic that touches an individual's health, and the sanitary engineer should have no difficulty in seeking entry to such groups in order to relate the strategic role of his work in the community's life.

These are but a few examples of the many ways by which an alert and ingenious sanitary engineer can make himself heard in a community. It is not necessary to be an oratorical "spellbinder." Audiences respond quickly to evidences of sincerity of motive, need for services, pride in professional work, concern for the welfare of others, and worthwhileness of message.

In brief, the problem of attracting more students to sanitary engineering may be met on many fronts by many people, and in many ways: analysis of student response, preparation of informational materials, provision for summer employment in the field, closer public school-university faculty relations, more strategic use of vocational counselors, closer liaison between professional engineering socieites and the community's social groups in general and the community's educational groups in particular, and the sanitary engineer's personal efforts to talk about his field, both as a professional and active citizen.

REFERENCES

1. As reported in Journal, Amer. Pub. Health Assoc., 47:1013 (Aug.'57).

2. Why Some Sanitary Engineers Leave The Field. Public Health Monograph No. 21 U. S. Public Health Service, Wash. 25, D. C. pp. 10.

3. Kendall, Nathaniel J., "Engineers in Short Supply," Journal, American Water Works Association, 49:470 (April, 1957).

4. See footnote 1.

Part III - Financial Aid

Higher education is expensive. Costs have risen in recent years for many reasons, some obvious and others not so obvious. High costs of living, needed expansion of university plant facilities, industry's lure to faculty people with attractive salaries — these are the more obvious explanations for higher tuition costs and higher total costs for degree-types of education.

With the price tags of higher education approaching prohibitive proportions, increasing numbers of students, the Nation's prime resource, either cannot or will not continue their education beyond the secondary school at the same time that the country's need for scientific and engineering talent becomes more pressing.(1)

Certain types of engineering education are becoming more expensive for professional as well as financial reasons.

Lengthening and Complexity of Engineering Education

With each passing year, more and more knowledge clamors for inclusion in the engineering curriculum. The trend is clear and unmistakable toward a basic, common denominator, four-year, undergraduate curriculum in the basic sciences, engineering sciences, and the social sciences and humanities, with engineering specialization taking place in the graduate fifth and later years. Many leading engineering schools either now offer or are seriously contemplating the requirement of a fifth year for the first degree.

The growing complexity of engineering knowledge and the increasing interrelatedness of many disciplines require both greater maturity and on-the-job exposure (while learning, if possible) in order to produce the professional engineer of quality.

Running counter to this lengthening period of training is the trend toward earlier marriage and larger families and the voracious appetite of industry for professional-technical personnel. The resulting need is obvious — financial assistance to many deserving student in some measure, for some period, from any and every source possible.

Lack of Funds as a Bar to Education

The lack of sufficient family funds to finance students through higher education is not new. The Roosevelt, Truman, and Eisenhower administrations, in turn, made serious and comprehensive surveys of higher education needs. These surveys all noted that, undoubtedly, many young people were attending colleges and universities who would either not profit from such exposure or would be better served by other types of institutions, such as junior colleges, trade schools, and technical institutes.

More important, however, was the persistent conclusion of these same surveys that thousands of bright young students who could profit from higher education were not able to get further education primarily because of the lack of funds. Reference is made to one such recent survey, undertaken by the College Entrance Examination Board for the National Science Foundation, to measure the loss of high level ability from secondary school to college. The report concluded that "insufficient financial support appears to be the sole or primary reason for the annual failure of between 60,000-100,000 students of superior ability to enroll in colleges."(2)

Once in college, the attrition or drop-out rate is well known. "Between 70 and 90 percent of our engineering freshmen become sophomores, with a normal figure in recent years of somewhere around 70 to 80 percent; between 60 and 75 percent become juniors; between 50 and 70 percent become seniors, with a normal figure around 60 to 65 percent."(3) A recent study of a sample of over 13,000 students indicates that the major reason for drop-out beyond the first year is lack of funds.

In view of the fact that sanitary engineering students constitute but one percent of the total number annually receiving their first engineering degrees, the lack of financial assistance can be catastrophic and support in some measure appears almost mandatory for this small group.

Competition Stimulates Financial Assistance

The notion that the scholarship winner is either a "long-hair recluse" or a "charity case" dies hard. The fact remains that the need for superior professional talents and skills has increasingly eliminated the insistence that the student work his own way entirely through college as a matter of principle "no matter how many years it takes!"

Such delays are considered wasteful of both time and talent. Financial assistance to simply the needy student who wants an education has now become a matter of financial assistance to the unquestionably bright student whose talent is sorely needed by our economy. The scholarship student works just as hard as did his counterpart of an earlier generation. He may work more efficiently, also, because in many cases "waiting on tables" has given way to part-time or summer employment in the actual field or industry of his professional interest.

As one prominent university president recently explained: ". . . higher education is not a charitable institution but an indispensable agency that must be given support to do its job."(4) In essence, the economy and way of life demand the cultivation of the Nation's best human talents, and financial assistance constitutes an inescapable investment in the future which reaps incalculable fruit.

Financial Assistance in Sanitary Engineering

The relatively small number of students (i.e., in proportion to the total professional engineering group) needed to fill the ranks in sanitary engineering permits emphasis on the quality of education and training. The completely graduate status of this specialty emphasizes the need for financial aid at a time when the student may be founding a family.

A majority of sanitary engineers are employed by public agencies at all levels (municipal, county, State, and Federal). Average salaries in the first years of professional service cannot match salaries in industry or the income of sanitary engineers in consulting work. In fact, increased competence on the job lead increasingly to employment in industry or independent consulting service. The relative rigidity of Civil Service and/ or merit-service salary schedules and the increasing needs of communities for the guardship and maintenance of many healthful environmental standards, provide clear demands on government for increasing support for the education and training of not only sanitary engineers, but also supporting and related specialists, such as sanitarians, chemists, biologists, bacteriologists, and physicists.

Recent years have witnessed the modest beginnings and steady growth of such support, both Federal and non-Federal. Aside from the famous Veterans Administration GI Bill, most such support has been at the graduate level. Pressures from industry and expanding community needs raise the question of the degree to which it may be desirable, or even necessary, to select certain worthy and talented students for support during undergraduate years as well.

Although the following data are incomplete, they do give an idea of the representativeness and comprehensiveness of support available to students who may select sanitary engineering for career emphasis. Major Federal financial aid programs will be described briefly and then the important non-Federal sources.

Federal Support

The United States Public Health Service administers four major programs, authorized by various Congressional action in recent years, for the purpose of stimulating both research and training in the fields of sanitary engineering and occupational health. These programs are (1) research grants, (2) research fellowships, (3) traineeships, and (4) training grants-in-aid.

Research grants are awarded to individual investigators for the support of research projects in sanitary engineering and related sciences principally to (1) expand research activities and (2) provide research training for personnel. During Fiscal Year 1957, more than 400 individuals shared almost $2,000,000, about 95 percent going to universities and another 4 percent to independent research laboratories.

The program of research fellowships is new and is supported initially on a modest scale in 1957. The fellowships are of three types in the broad field of sanitary engineering — predoctoral, postdoctoral, and special. Applicants may have basic training background in engineering or related biological, chemical, and physical sciences. The primary purpose of this program is to increase the number of engineers and scientists qualified to conduct independent research in problems of environmental sanitation Five such fellowships were awarded during 1957, each worth $1,800 plus tuition plus a family allowance and totaling an average of over $4,000 It is planned to double this number of fellowships in the coming year.

The traineeships, under Title I of Public Law 911, are available to sanitary engineers, sanitarians, sanitary chemists, and allied personnel wishing to study in a graduate program having definite public health implications. They are intended to bring new people into the field of public health and thus give preference to those with no more than two years' experience in public health, who have had less than one year of graduate or specialized public health training after obtaining their professional degree, and who are under 35 years of age. Approximately 130 traineeships were awarded in the past several years to sanitary engineers, sanitarians, chemists, physicists, industrial hygienists, biologists, and geologists. The average stipend is $4,000.

Training grants-in-aid are awarded under the Air Pollution Research and Technical Assistance Act of 1955. So far, some 14 individuals have been awarded average stipends of $4,500 for the purpose of increasing their competence in the prevention and abatement of community air pollution. Eleven grants were made directly to certain educational and training institutions to assist in developing graduate level study programs in air pollution,

and three grants were awarded to air pollution control agencies to pay for the specialized training of certain employees.

All the programs described here, with the exception of the research grant activity, are quite new, show great promise, disclose mounting interest, and are slowly expanding. Indication of potential growth may be noted from the actual development of the research grants program. In 1947, six grants totaling $66,284 were awarded in the field of sanitary engineering and occupational health. Ten years later, 73 such grants were awarded, in 1956, totaling $880,853.

Other Federal Agencies

Other Federal agencies provide stipends of various categories for study in a great range of fields, though only a few are peripheral to public health interests as such. The Veterans Administration, National Science Foundation, Atomic Energy Commission, and Department of Defense are the major providers. Other than the famous GI Bill of the Veterans Administration, the financial aid programs of the other Federal agencies are heavily weighted toward postdoctoral research. Even the Public Health Service lends its greatest support in the health field to medical research through grants administered by the National Institutes of Health.

Of particular interest is the program of special fellowships under the aegis of the Oak Ridge Institute of Nuclear Studies. Study leading to the Master's degree in industrial hygiene is supported by $2,500 grants for 10 months plus tuition, other fees and family allowances. Graduate predoctoral fellowships of the National Science Foundation are available to individuals desiring to pursue graduate work in the physical, biological, engineering, and related sciences. Stipends range from $1,400 to $1,800.

In the past few years the Federal Government expended altogether about $50 million in aid to some 43,000 graduate students in 1954-55. Almost half of these were studying in science fields. The largest group of graduate students were employed as research assistants and were working on research contracts or grants awarded to senior investigators at colleges and universities.

Since the emphasis is on research and teaching, Federal funds available for graduate students planning to practice their profession on the job in various communities are, indeed, limited. More, much more, needs to be done in this particular Federal area of assistance.

To this end, it is pertinent to note that a Conference on the Education, Training, and Utilization of Sanitary Engineers was held this past spring in Washington, D. C. The two-day meeting was conducted under the auspices of the Subcommittee on Personnel and Training of the Committee on Sanitary Engineering and Environment, which is a unit of the National Academy of Sciences, National Research Council.

Outstanding educators, engineers, and administrators focused their attention upon a number of problems, including that of financial aid, relating to sanitary engineering personnel and training. Recommendation was made in strongly affirmative tone that graduate-level fellowships and traineeships should be expanded and should carry no restrictions on the duration of training awards to any qualified individual up to and including the Doctoral program. The conference was equally forthright and insistent in its recommendation that Title I of Public Law 911 should be expanded to include financial assistance for the undergraduate education of engineers, and should provide

a monthly stipend of not less than $100 plus an allowance for tuition and books.

Non-Federal Support

Financial assistance to sanitary engineering graduate students is quite meager in institutions of higher learning. In a survey made recently, by the authors, of all institutions of higher education known to offer training in sanitary engineering, very few provided assistance in this field. Though the data are not exhaustive, they are representative of the nature and extent of aid currently available to graduate students in this field.

A total of 17 schools, three foundations, and four professional organizations offer graduate students scholarships and/or fellowships in which the study of sanitary engineering is permissable. These schools include the Massachusetts Institute of Technology, California Institute of Technology, Virginia Polytechnic Institute, Rensselaer Polytechnic Institute; the Universities of California (Berkely), Oklahoma, Utah, Washington, Florida, and Michigan; Northwestern, Rutgers, Harvard, Johns Hopkins, and Oklahoma State Universities. This group provides an average of 47 separate stipends, ranging in value from $400 to $4,500 per year. It must be noted that the awards in a few of the above-named universities are closely linked with graduate research requirements.

The American Water Works Association, the American Public Works Association, the National Lime Association, and the National Council for Stream Improvement, each offer a graduate stipend, with value up to $1,500, $1,000, $2,400, and $2,574 per year respectively. These are specifically for sanitary engineering. The Dorrco Foundation (Rutgers U.) offers a stipend of $2,400 leading to the Ph.D. The Kemper Foundation (Harvard U.) is limited to study in industrial hygiene and carries a sum varying with need and project. The Clow Foundation in the midwest offers a sum up to $1,800 per academic year to a graduate of a college or university in the states of Ohio, Michigan, Illinois, Indiana, Iowa, or Wisconsin. This award is for graduate study in sanitary engineering leading to the Master's degree.

Another sixteen colleges and universities reported the availability of financial assistance. In general, such money is offered in part payment for services rendered in the form of membership on a research team or part-time teaching, and are labelled "research assistantships" or " teaching fellowships." Actually, most colleges and universities with graduate programs provide such remuneration, if only of a token nature, in exchange for the "junior faculty" services rendered to overburdened professional teaching or research faculty.

Despite the importance of such assistance, the focus is on the professional teaching or research engineer and not on the young engineer who is to be trained for a career in the community or in industry. Much more attention must be given to this latter group.

The authors urge recognition of the fact that a graduate engineer can, if he chooses, find full-time employment in the professional field in preference to graduate study; this at a time when he may be assuming personal and family obligations; that he is a member of a strategic and elite group of engineers whose duty — and privilege — it is to guard the community's health; that there are few such professionally trained individuals; that most of them are public service employees whose salaries, while adequate, are

meager when compared with those of similar positions in industry; that advanced, graduate, professional education and training must match the increasing complexities and responsibilities of sanitary engineering; and that when an outstanding young individual is identified, and indicates his willingness and aptitude for this field – for all these reasons, it becomes mandatory that agencies and institutions, both in and out of government, provide every possible financial assistance to such young people.

The sum should be a significant one, substantial enough to reflect investment rather than charity. The thirty or more years of professional service to be rendered will benefit more than just one company, or even one community. Hundreds of thousands of people are involved. The facilities the sanitary engineer provides and maintains are not optional to the public — they are, many of them, absolutely vital to our very existence. The community has a right to the most wholesome, healthiest environment it is possible to achieve. This is both a public and private right. Both government and industry, then, have the inescapable obligation of guaranteeing the greatest possible competence in those individuals who elect to apply their skills and talents to extending and maintaining the strategic services everyone wants and needs.

Government, industry, and the academic world have made attempts to provide significant financial assistance to graduate students in sanitary engineering. These efforts are well-intentioned, genuine, and modest. They are not enough. They must be expanded substantially.

REFERENCES

1. See editorial, "Costs of Higher Education," Science, pp. 126:333, August 23, 1957.

2. Norman N. Barish, Engineering Enrollment in the United States, New York University Press, 1957, p.51, derived from Charles C. Cole, Jr., Encouraging Scientific Talent, New York, College Entrance Examination Board, 1956.

3. Norman N. Barish, op. cit., p. 14.

4. Gene Currivan, "Education In Review," New York Times, September 1, 1947. 1957. P.E. 9.

Financial Assistance Available in Institutions of Higher Education, Professional Organizations, and Foundations for Students Interested in Sanitary Engineering, 1957

A. Full Support

SOURCE	TYPE	SUPPORT	STIPEND	REMARKS
California Institute of Technology	Tuition Grants	University		
California, Univ. of	Scholarship (2)	James Monroe McDonald Scholarship	$400/yr.	Only Sanitary Engineering. No tenure limitations.
	Scholarship (7)	University	$600-1500/yr.	No field of specialty.
Florida, Univ. of	Fellowship	College of Engrg.	$1800/yr.	No specialty.
Harvard University	Research in areas related to Cl. of water	Wallace & Tiernan	$2000/yr.	Particularly to students in Engrg. Health Service
	Scholarship	Genge C. Wipple Scholarship	Varies (Abt.$825 for 1958-59 year)	In Sanitary Engineering.
	Kemper Fdn. Fellowship	Lumberman's Mutual Casualty Co.	Varies with need & project	For training in Industrial Hygiene.

SOURCE	TYPE	SUPPORT	STIPEND	REMARKS
Harvard University	AEC Fellowship	AEC	Tuition & Stipend depend upon family obligations and experience	For training in Industrial Hygiene.
Johns Hopkins University	Fellowship	Salem-Brocios Inc.	$4500/acad.yr.	Ph.D. Candidate.
Massachusetts Institute of Technology	Fellowship	Tau Beta Pi	$1250/acad.yr.	For one year.
	Fellowship	Whitney Fellowship (MIT)	$1000/acad.yr.	For one year.
	Fellowship	Solar Energy Fund Fellowship (MIT)	$2400	For two years.
Michigan, Univ. of	Research	Industry	$1500/acad.yr.	No limitations. No specialty.
	Research	University grad. fund	$1800/acad.yr.	No limitations. No specialty.
Northwestern University	Fellowship	Institute Fellowship	$1800/acad.yr. (Tuition not included)	M.S. or Ph.D.
	Tuition Scholarship	University	$660/acad.yr. (full tuition)	M.S. or Ph.D.

SOURCE	TYPE	SUPPORT	STIPEND	REMARKS
Northwestern University	Fellowship	Cabell & Baker Fellowships	Up to $3600 per acad.yr.	
Oklahoma State University	Fellowship	University	$1710-1845 per acad.yr.	One year tenure.
Oklahoma, Univ. of	Fellowship	John Dairy	$75/month	
	Fellowship (2 students)	Oklahoma City	$125/month	
Purdue University	XR Fellowship	Research Fdn.	$2000/yr.	For Ph.D. only
	Fellowship	Engrg. Experiment Station	$3600/yr.	M.S. or Ph.D. level
Rensselaer Polytechnic Institute	Scholarship & Fellowships	Institute	N. A.	No specialty.
Rutgers University	Fellowship	Dorrco Educational Foundation	$2400	Ph.D. training - 4 tenure limitation
	Fellowship	National Lime Assoc.	$2400	Ph.D. training - 4½ tenure limitation
	Fellowship	National Council for Stream Improvement of Paper, Pulp, & Paperboard Industry	$2574	Ph.D. training - 4½ tenure limitation

SOURCE	TYPE	SUPPORT	STIPEND	REMARKS
Utah, Univ. of	Fellowship	Utah Exp. Station	$900/acad.yr.	
Virginia Polytechnic Institute	Fellowships (1 to 4 per yr.)	School	$1035-1560 (9 or 12 Mo.) (Pay own tuition)	Can be held for 1 yr. if working for M.S.; or 2 to 3 yrs. if working for Ph.D.
Washington, Univ. of	Fellowship	Engrg. Exp. Station	$150/month (tuition exempt plus $300-400 for equipment & supplies)	One year tenure on M.S. Program.
American Pub. Wks. Assoc.	Fellowship	Engrg News-Record Aid-to-Education Award	Not to exceed $1000	Study in field of P.W. Engrg. and/or Administration.
American Water Wks. Association	Harry E. Jordan Scholarship Award	Water & Sewage Works Manufactuers Association	$1500/yr. for one student (may be split)	Limitations on area of competition. Purpose is to contribute to the advancement of water supply practice.
James B. Clow & Sons, Inc. (through Clow Foundation)	Fellowship	James B. Clow & Sons, Inc., through Clow Foundation	Up to $1800 per acad. yr.	Available to a graduate of college or univ. in the states of Ohio, Michigan Illinois, Indiana, Iowa and Wisconsin. Grad. study in san. engrg. leading to M.S. degree.

B. Support which includes part-time services

SOURCE	TYPE	SUPPORT	STIPEND	REMARKS
Alabama Polytechnic Institute	Teaching	School	$1260/acad.yr.	Full graduate program permitted.
California Institute of Technology	Tuition Grants	University		
California, Univ. of	Half-Time Grad. Students (2)	Rockefeller Foundation	Depends on research project	Research contract.
	Half-Time Grad. Student (1)	Assn.of American Soap & Glycerine Mfg.	Depends on research project	Research contract.
	Half-Time Grad. Student (1)	U.S. Bureau of Reclamation	Depends on research project	Research contract.
	Half-Time Grad. Students (1)	A.E.C.	Depends on research project	Research contract.
	Half-Time Research (1 to 3)	State of California	$1800-2400/acad. year	Grad. study at the doctorate level

SOURCE	TYPE	SUPPORT	STIPEND	REMARKS
California, Univ. of	Half-Time Grad. Students (1)	U.S. Army Engineers	Depends on research proj.	Research contract.
	Half-Time Grad. Students (5)	State of California	Depends on research proj. (From $1800 to 2400/yr.)	Research contract.
Case Institute of Technology	Graduate Assistantship	School	$2200/yr ($500 deducted for tuition)	Half-Time Services Required.
Cornell University	Teaching Assistantship (Available 9/1/57)	University	$3200/acad.yr. (Tuition & fees of $1000 must be deducted from stipend)	30 hr. week assisting in Sanitary Engineering.
Florida, Univ. of	Research Assistant-ship (2 to 3)	University	$210/mo.-2/3 res-idence cred. $155/ mo.4/5 res.cred.	
Harvard University	Teaching	School	$1000/acad.yr. ½-time work $2000/acad.yr. ½-time work	General award
	Assistantship (2)	Research Contract	$1800-2100/acad. year	A.E.C. Research. Summer employment opportunity.

SOURCE	TYPE	SUPPORT	STIPEND	REMARKS
Harvard University	Assistant-ship (2)	Am. Iron & Steel Inst. Grant	$1800-2100/acad. yr.	Air Pollution Res. Summer employment opportunity.
Iowa State College	Grad. Assistant-ship (2) (1 water supply, 1 sewage disposal)	School	$1350-1500/acad. yr.-½ time (Fee reduction)	M.S. or Ph.D. candidate.
Johns Hopkins University	Assistantship	School	$1800/acad. year	Ph.D. candidate.
Kentucky, Univ. of	Graduate Assistantship		$375/month	2 yrs. to get M.S.
Maine, Univ. of	Half-Time	Research	N.A.	
Michigan, Univ. of	Research Assistantship (4)	Municipality	$1700	
	Research Associate (1)	Municipality		
Missouri School of mines & Metallurgy	Assistantships	School	$1500/acad.yr.	Serve as part-time laboratory teachers.
Missouri, Univ. of	Research	College of Engrg. Exp. Station	$1800 for service 15-hr. wk. serv. (12 hr. sem.study poss.)	(No tuition chg.)
			$1200 for service 10-hr. wk. serv. (14 hr. sem. study poss.)	(No Tuition chg.)
			$900 for service 8-hr. wk. serv. (16 hr. sem. study poss.)	(No tuition chg.)

SOURCE	TYPE	SUPPORT	STIPEND	REMARKS
Missouri, Univ. of	Teaching	University	$1800 for service 15 hr. wk. serv. (12 hr. sem.study poss.)	(No tuition charge)
			$1200 for service 10 hr. wk. serv. (14 hr. sem.study poss.)	(No tuition charge)
			$900 for service 8 hr.wk. serv. (16 hr. sem.study poss.)	(No tuition charge)
Newark Coll. of Engrg.	Asst. Instructor	School	$4000-5000	Full time day teaching & 1 evening/wk.(extra pay) Full tuition recd. for grad. work in San. Engrg.
North Carolina, Univ. of	Half-Time Assistantship (2)	University	$1500/acad.yr plus	Plus $1050 for full time summer work.
	Assistantships (4-6)	Research Contracts	Varies	Complete MS or MPH in 2 yrs. in san. engrg. san. science, san. chem. & biology.
Northwestern University	Teaching Assistantship	University	$2000/yr.(Full tuition of $880/yr included)	Study for M.S. or Ph.D. One year appt.
	Research Assistantship	University	$2,000/yr.(Full tuition of $880/yr. included)	Study for M.S. or Ph.D. One year appt.

SOURCE	TYPE	SUPPORT	STIPEND	REMARKS
Northwestern University	Research Assistantship	Research Project	Depending on project	
Ohio State University	Research (2)	University	$1200/acad.yr.	Renewable.
	Teaching Assistantships	University	$1800/acad.yr. (Exempt out-of-State fees)	Teaching part time.
Oklahoma, Univ. of	Research Assistantship (1 student)	GE Compost Contract	$125/month	
	Research Assistantship (2 students)	Dick Clay Pipe Association	$50/month	
	Research Assistantship (2 students)	U.S. Air Force	$125/month	
Rutgers University	Assistantships (3)		$1830 plus Tuition/acad.yr.	No specialty; 2 yr. tenure limitation, an. appt. renewable for 1 add. final yr. for M.S.; req.15/hr. wk. service.
Tennessee, Univ. of	Instructor	School	$100/credit hr. taught per term (pay no tuition)	Take 6 hrs. grad. work per term

SOURCE	TYPE	SUPPORT	STIPEND	REMARKS
Tennessee, Univ. of	Instructor (3-5)		No tuition $150/cred.hr. per term pay.	
Texas, Univ. of	Traineeship (2) Traineeship (2) Traineeship (1) Traineeship (2) Traineeship (2)	University University University University University	$183/mo. $\frac{1}{2}$ time $150/mo. $\frac{2}{3}$ time $500/mo. $\frac{1}{2}$ time $75/mo. $\frac{1}{4}$ time $125/mo. $\frac{1}{4}$ time	
State College of Washington	Research Assistantships (3)	Div. of Industrial Research	$1650/acad.yr.	
Wisconsin, Univ. of	Research Assistantship (2 available 1957-58)	Wisc. Alumni Research Fdn.	$160/mo. (allow. for supplies)	Exempt from paying out-of-State tuition fee.
	Assistantships (3)	Research projects	Vary with project	
Wyoming, Univ. of	Fellowship (2)	School	$990/acad.yr. (Remission of fees)	Require 15 hr./wk. of service.

NOTE: The data in this table are believed to be comprehensive but not necessarily complete and exhaustive.

Paper 1714 July, 1958 SA 4

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

FRINGE AREA SEWERAGE PROBLEMS

John E. Kiker, Jr.,[1] M. ASCE
(Proc. Paper 1714)

ABSTRACT

The paper reviews the problems and some of the means of solving fringe area sewerage problems. In the discussion the relative merits of septic tanks, package plants, and oxidation ponds are considered from the standpoint of construction and operation.

INTRODUCTION

Some of the most authoritative information about Florida is contained in a well-known publication which cannot be named because of its copyright and inflexible "no quotation" clause. However, people who read the publication are impressed by its accuracy and enlightened comments. It calls a spade a spade and does not pull punches where facts are involved. Of course, most of the information on Florida is optimistic and encouraging. In some respects, however, the news about Florida is not all good.

An example of this was brought out in an issue dated March 15, 1958. In addition to a paragraph on Florida's silver lining, there was a section pointing out some bad signs. Among them were three which may be paraphrased as follows:

1. Septic tanks will not work because the water table is too close to the ground surface and there is not enough drainage.
2. Some housing developments have been held up because of septic tank problems.
3. Land which was previously thought suitable for housing developments is now wet. Pools of surface water are standing around the houses, discouraging buyers.

Note: Discussion open until December 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1714 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 4, July, 1958.

a. Presented at the 29th Annual Meeting, Georgia Public Health Association, Savannah, Georgia, April 14-16, 1958.

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After reading such comments one is forced to conclude that they are just about as good a vindication as could be given of the position maintained for years by the Chief of the Bureau of Sanitary Engineering of the Florida State Board of Health in discouraging the use of septic tank systems in fringe areas. Many sanitarians and sanitary engineers will recognize that the statements are accurate for certain sections of the country outside of Florida. Change the words "the water table" to "impervious clay" in the first sentence and the sentence will apply to much of Georgia.

The second sentence will apply to every state in the union; more in some states than in others of course, but it is true nevertheless.

The third statement may apply peculiarly to Florida and only to certain sections of other states, but it is believed that some of the other states are having about as much trouble as Florida even though their septic tank failures may not be due to wetness or "surface water". According to published statements of a Public Health Service regional engineer, for example, less than half of the septic tanks are working satisfactorily in a five-state area of the Midwest, including Illinois, Indiana, Michigan, Ohio, and Wisconsin.(1) This is truly an indictment of sanitary engineers and sanitary engineering, and it is time more sanitary engineers started trying earnestly to correct the situation. One purpose of this paper is to help in that direction.

Septic Tank Failures

The reasons for failures of septic systems are well known. They have been published and discussed, so often—by the writer and others—that there is little technical information to be added to the subject at this time. Some were mentioned in a discussion of "New Developments in Septic Tank Systems" given at the annual meeting of the Georgia Public Health Association in 1956. The discussion was published in the October 1956 issue of the Proceedings of the American Society of Civil Engineers (Sanitary Engineering Division)(2) and it will appear also in the ASCE 1958 Transactions. Following that it will be reprinted in the technical paper series of the University of Florida Engineering and Industrial Experiment Station and copies will be made available to anyone who wishes them. (For the meantime, a few mimeograph copies are available on request).

Manual of Septic Tank Practice

Most of the information contained in that paper, and much more, is included in the Manual of Septic Tank Practice(3) published by the U. S. Public Health Service in 1957; although the manual itself does not include the reasons for some of the new standards that were explained in the paper. Again, it is not considered desirable to repeat here the information contained in the manual, but it may be well to explain a few more things about the manual, based, this time, upon reactions from the field about some of its contents.

To start with, however, it should be apparent to any thinking person that it would be manifestly impossible to develop a manual that will satisfy everyone. Most people seem to realize this and the reception of the publication to date has been excellent. But it has been criticized on some points, and as one of its authors receipt has been had of a number of complaints and inquiries. No doubt the PHS representatives who participated in its preparation have

received even more. In this discussion, the compliments will be disregarded and only the complaints considered. Some of the criticisms have been valid and should help in any revision to be undertaken. Others have not been valid. A few of both will be discussed.

There has been some objection to the make-up of the Joint Committee on Rural Sanitation which lent advisory assistance in the preparation of the manual. One friend expressed it simply by stating that the committee was "loaded". He would have liked more "grass roots" representation, and the point is believed to have been well taken. As one who worked on the manual, however, it is also thought that the committee was much too large. Including the industry advisors, no less than 27 persons were named in the Foreword as having participated in the preparation. These included 18 federal employees from 12 different governmental agencies. But how could you expect the different federal agencies concerned with septic tank systems to accept the manual as a working guide unless they were represented on the committee?

Interpretations

Another objection has been that the standards recommended in the manual are too high and that they will practically price septic tank systems out of existence in fringe areas. Time and space will not permit a complete answer to this objection. Suffice it to say that one of the main purposes of the manual is to help in the prevention of so many failures of septic tank systems. In spite of the fact that there have been altogether too many failures, it is also true that in some jurisdictions where there has been proper supervision and enforcement of the old standards, relatively few failures have occurred. The manual was not written for the engineers and sanitarians in areas where the record of such an accomplishment has been good. It was written more for the people who require help; to give them the tools and the kind of backing that is needed to prevent future failures. If there be any experienced health officer, or engineer, or sanitarian who is so satisfied with the old standards—or is so convinced as to their adequacy that he is willing to stake his reputation on continued adherence to them—then by all means that individual should be permitted to do so. But let him not decry the opportunity that is now being offered the less fortunate areas to put a halt to the failures.

Perhaps a more valid complaint about the manual might be that the standards are not high enough. In Florida, for example, seepage pits are not legally permitted under any circumstances, and subsurface tile field disposal areas are permitted only where the drop in water level during percolation tests remains greater than one inch in ten minutes after "saturation". Now there are perfectly good reasons for these special standards. Even though many people from without the State of Florida consider them too high, experienced engineers within the state have found that they are not high enough. And no person from without the state who is inexperienced in Florida conditions has a right to challenge the things that enlightened and conscientious Floridians believe best for Florida.

Conversely, Florida engineers should have no right to quarrel with the New York State practice of permitting seepage pits in soil where percolation rates are as slow as one inch in 60 minutes. In spite of the fact that the Septic Tank Manual recommends a minimum percolation rate of an inch in 30 minutes, New York engineers should not be expected to change their standard

until they find there is a need for so doing. But the Septic Tank Manual could be a useful tool in backing their position if they wish to make a change.

Explanations and interpretations of this sort could continue ad infinitum. They would apply equally to the 2-1/2" versus a 2" minimum wall thickness for precast reinforced concrete septic tanks; as well as to depth and width ratios, to tank capacities, to absorption areas, to the use of enzymes and other proprietary products for which extravagant but unsubstantiated claims have been made, to the methods of making percolation tests, and to countless other questions on which there will never be universal agreement. It should be repeated, however, that the manual could be a useful tool in the solution of many if not most of the septic tank problems.

Answers

It is becoming increasingly apparent to many if not most sanitary engineers that "the only satisfactory way to dispose of domestic sewage is by public sewers with subsequent adequate treatment", and that an outstanding deficiency leading to sewerage problems in fringe areas is lack of planning.(4) According to a recent article based on a report of a New York State Public Health Association committee studying the problem of providing public sewerage in realty subdivisions, all committee members but one were definitely opposed to septic tank installations in realty subdivisions in built-up or potentially built-up areas, regardless of soil and ground conditions.(4) The report also indicated that septic tanks should be reserved for use in predominantly rural areas. If this be true, and increasing numbers of engineers are beginning to consider it so, what is the answer? There is no single or simple answer. But a great deal of progress is being made in solutions to the problem. One such solution lies in the use of "package" or prefabricated sewage treatment plants.

Package Plants

Although agreement on the point is not complete, some engineers feel ". . . that the term 'package sewage treatment plant' should be used only for an 'off-the-shelf' unit which is completely fabricated at the factory and shipped to the site complete except for the necessary connections."(5) Also that any other variation should be classified as a "small sewage treatment plant".(5) This seems a good definition and will apply in the remarks which follow.

A very good article on small community plants appeared in the February 1958 issue of HOUSE AND HOME magazine.(6) It mentions the names of three prefabricated plant manufacturers from whom additional information was requested for the preparation of this paper. When the Director of the Bureau of Sanitary Engineering of the Florida State Board of Health learned that this request had been made he very graciously wrote a letter which included the following admonition: ". . . I would like to caution you with reference to all these package plants because if we get involved with them we will end up with a lot of these aerobic propositions which will leave us with a lot of monuments. I think we had better stay in the field of orthodox type of plants that can be operated." There, in a nutshell, is the kind of information which should be seriously considered before any package plant is approved. Although it does not say so directly, it implies that some kind of the prefabricated aerobic

treatment devices require competent operation of a kind that is not commonly available for small community plants.

"Household Plants"

The need for proper operation of "aerobic treatment devices," even at the household level, was brought out rather facetiously—but also quite dramatically—last month by the Executive Secretary of the Federation of Sewage and Industrial Wastes Associations. In a paper delivered on March 11, 1958, at the University of Florida during a conference on Metropolitan Planning, he had the following to say in a paragraph headed "Aerobic Treatment Devices":

> "Limitation in provision of sewer services in fringe areas is that the cost of sewer extensions can become unreasonable with spotty development. This has caused many developers to turn to individual household sewage treatment rather than the provision of the community sewer system. While the septic tank has been a traditional approach, this unit now finds itself in competition with more complicated devices which I choose to call aerobic individual household sewage treatment plants. These are now being offered by reputable firms and it gives the householder an opportunity to become his own sewage treatment plant operator if he wishes and if he can obtain clearance of health authorities. In spite of my life long interest in the technicalities of sewage treatment, if permitted, I would prefer an oversized septic tank with annual inspection and cleaning each five years or more frequently. I express this choice in deference to my desire for Federation growth with a copy of SEWAGE AND INDUSTRIAL WASTES in every living room each month."(7)

With that illuminating quotation, it is hoped that any question concerning the use of aerobic individual household sewage treatment plants may be considered answered.

Community Plants

As regards the use of aerobic package plants for small communities, it is suggested that they not be considered unless competent operation can be guaranteed. The quality of such operation should approach that required for an activated sludge plant, and this should rule out consideration for most small communities.

In partial substantiation of this opinion, there was an excellent article on "'Packaged' Treatment Plants" in the July 1957 issue of MODERN SANITATION.(8) It was reprinted in a brochure entitled "Coping with Suburban Sewage Disposal Problems", which was prepared and released in March 1958 by the Conference of Municipal Public Health Engineers. The author was Edward Stockton, Director, Bureau of Sanitation, Calhoun County Health Department, Battle Creek, Michigan. As indicated by Stockton, mechanical aerators and other aeration equipment were offered by several manufacturers in the mid 30's, with extravagant claims for their adaptability and superior performance for small communities and institutions. Performance was disappointing, however, and public health officials became reluctant to accept any more new equipment without sufficient testing in advance. Much of the apprehension still exists about equipment recently developed.

Because of this situation, the Michigan Public Health Association appointed a committee of engineers to study the problem of small sewage treatment plants. The committee contacted some 20 state health department engineers and several equipment supply firms. It concluded that data in some instances are not sufficient to establish predictable performance levels. Some of the deficiencies include tests on grab samples instead of composite samples, an insufficient quantity of data, inadequate information on flow rates, lacking information on power requirements, equipment maintenance, repair and replacement, and lacking information on the extent to which plants are loaded as compared with design loadings. The committee suggested that additional research be conducted by some unbiased organization similar to the National Sanitation Foundation.

The committee also suggested that the proposed testing and evaluation program be restricted initially to equipment and processes being developed for small, non-public installations. Then, if the plants prove as effective as preliminary reports indicate, they should be accepted for small public installations. This is thought to be a reasonable procedure. Although every sanitary engineer wishes to encourage progress, those in public positions have an obligation to protect the taxpayers from losses which would be incurred through the installation of equipment or treatment facilities which may not accomplish the purpose intended.(9) Engineers in industry also have obligations to their employers, but it is sometimes easier for one industry to sell a new product to another industry than to a governmental unit. And, ordinarily, a state or local sanitary engineer would be much more likely to sanction the installation of one of the unproven treatment devices for an industry which can be held responsible for the results of treatment, than for a realty developer who might not be around any longer than is necessary for him to collect his profits. Of course, there is much more to this problem of responsibility or operational accountability than is covered in these simple statements, but it should be apparent that a sanitary engineer will usually acquiesce more readily in accepting an unproven treatment unit if he does not also have to share in the responsibility for its successful operation. The sooner manufacturers realize this, and themselves become willing to assume reasonable guarantees and responsibilities for the successful operation of their products, the sooner will real progress be made. This responsibility cannot be discharged by advertising that their somewhat complicated devices are practically automatic or even that they do not require competent operating skills.

Another thing not adequately appreciated by some manufacturers' representatives is that although they may have the very best of the new treatment devices, their competitors will not acknowledge this. The competitors put up just as good a story about their products, and the sanitary engineer who lets the bars down for one is placed in something of an untenable position if he does not do so for another. Once he starts, his problems begin to increase. Multiply the first installation by ten, or a dozen, and then by a hundred, perhaps, and the sanitary engineer finds himself in real trouble.

Stage Construction

The problem, however, is not as bleak as it might appear at first glance. There are some signs of hope, and even of accelerated progress. For one thing, while waiting for further proof on the more complicated of the package

plants involving mechanical aeration devices and high quality operation or maintenance skills, the desired end may be accomplished in some instances through stage construction of plants of the more orthodox or proven designs. Plants can be designed so that treatment units serving a first group of houses can be converted to other uses and combined with additional treatment units when additional houses are built. In other words, plants may be designed in two, three, or more stages to handle the volume of sewage and give the degree of treatment required.

Because so much of Florida's new population is settling in fringe areas, the state has become something of a pioneer in stage-developed sewage treatment plants. For example, in a development at Pompano Beach, Florida, a 3-stage plant was designed. The first stage consisted of an Imhoff tank, a standard-rate trickling filter, a lift pump, a final settling tank, and a sludge drying bed. Provision was made for the addition of chlorine to the effluent line from the final settling tank. These facilities provided treatment for the first 125 homes, at a total cost of $75,000, or $600 a house.(6)

The second stage provided merely for one additional unit—a combination or split primary and secondary clarifier. The Imhoff tank was to be converted to a sludge digestion tank, the low-rate trickling filter to a high rate filter, and the secondary settling tank to a chlorine contact tank.

The third stage provided for the addition of another secondary clarifier, and conversion of the split primary-secondary clarifier to a single primary clarifier. Actually, stage 2 was never built because the development proceeded so fast that it was not needed, but the planning shows the kind of thing that can be done when a developer is limited in his initial investment. Stage 2 was designed to serve 250 homes; and stage 3 for 500. Stage 3 cost an additional $65,000, bringing the cost per home down to $280.

Proven Treatment

At this point, mention should be made of types of package plants which involve only simple or proven methods of treatment and do not include the more complicated mechanical devices or, rather, those which do not require higher quality operation and maintenance. Several manufacturers offer package plants which involve relatively orthodox treatment methods, and these may be more readily acceptable at this time.

One is the so-called "CompleTreator"[2] which incorporates in a single unit what would normally require five separate tanks in large scale installations. These include primary and secondary clarifiers, primary and secondary high-rate trickling filters, superimposed upon a sludge digester. This kind of plant has been recommended for 50 home subdivisions and at last report the complete cost is around $25,000 installed; averaging about $500 per house.

Although not a "package plant", as the term was defined earlier, a so-called "Spiragester"[3] is a combination primary clarifier and sludge digestion tank. It has been described as a modern version of the Imhoff tank, and is adaptable to subdivisions in fringe areas where primary treatment is adequate. For complete treatment, the Spiragester may be followed by a trickling filter and

2. Manufactured by Dorr-Oliver, Inc., Stamford, Conn.
3. Manufactured by Yeomans Brothers, Chicago, Ill., and called a "Spirahoff" in Florida.

final clarifier.[5] These plants are engineered and the equipment for them is fabricated to meet individual needs, but costwise and otherwise they are considered competitive with true package plants in the usual range for subdivisions.

Costs

In some of the references listed with this paper, a considerable amount of data is given on the relative costs of septic tank systems as compared with the costs of small sewage treatment plants including package plants. Based on comprehensive studies sponsored by the Public Health Service, Richards reports that the approximate average costs of septic tank installations will fall within a range from $150 to $500.[10] He quoted the approximate cost to be $150 in Florida; $250 in New York; $300 in Oregon and Texas; $350 in New Jersey, Illinois, and Missouri; $400 in Kansas; and $500 in Indiana. He gave figures showing that small community plants can frequently be built for comparable costs, although they vary from much higher costs in some cases to smaller costs in others. Fuhrman has given additional substantiating data in indicating that costs in many instances would seem to be equivalent, but he points out that there is vastly more satisfaction to the resident where community sewerage facilities are available.

The cost data reported to date, however, have been based upon the old minimum standards before the adoption of those recommended in the new Manual of Septic Tank Practice. Future comparisons should greatly if not overwhelmingly favor public or community sewerage facilities in most cases, because adoption of the new standards will generally increase the cost of septic tank systems from 50 to 100 per cent or more. Comparisons of costs of systems on the basis of the design criteria recommended in Bulletin No. 23 of the Florida Engineering and Industrial Station, with the costs of systems designed as recommended in the new manual, will illustrate this point.[11]

In the case of a 3-bedroom house, where a drop of 1 inch in 5 minutes is obtained during percolation tests, the required absorption area will be increased from a minimum of 175 to 375 sq. ft. and the net tank capacity will be increased from a minimum 600 to 900 gallons.[4] Using the criteria in the Florida State Sanitary Code, the values would be 75 sq. ft. and 600 gallons, respectively. The low absorption area requirements in Florida is suggested as one important reason for so many failures in this state.

Oxidation Ponds

Being a resident of Florida, the writer hesitates to mention lagoons, stabilization ponds, or oxidation ponds, for the reason that they are not generally acceptable in Florida. One of the reasons for this is because of Florida geology and the presence of numerous sinkholes leading to cavernous limestone formations through which sewage pollution has reached public water supplies. Oxidation ponds have proven their worth in other states, however, and they

4. Actually, the old and new figures would be closely comparable if the old recommendations concerning allowances for garbage grinders and other household appliances were followed. In practice, however, this was seldom done.

have been gaining favor since 1954 or earlier. Where they are properly designed and constructed, and are not overloaded, they provide good treatment and give little or no odor problem. Their operation is simple and their cost of maintenance low. In general, they provide the most economical method of sewage treatment.

One big advantage of lagoons is that they can be readily abandoned and their site used for other purposes when the area becomes built up or served by new facilities. For example, the site could be used for a park, or a playground, or even for a regular sewage treatment plant, when the area becomes built up and land prices increase.

Recommended Design Features

Area: 25 to 50 homes per acre of pond area.

Buffer zone: At least three times the lagoon area. As a minimum, several hundred feet should encircle the lagoon.

Dikes: Height 8 ft., berm 10 ft., inside slope 1 ft. vertical to 4 ft. horizontal.(12)

Net liquid depth: 3 to 5 ft.

Miscellaneous: Inlet should discharge near pond center and bottom. Locate outlet at edge, as far from inlet as possible.

CONCLUSION

> "As someone on a witness stand put it, the construction of hundreds of septic tanks in a high class residential subdivision is about like a person walking down the street with a silk hat on his head and a hole in the seat of his britches."(13)

When cost comparisons are made between septic tank systems and community sewage treatment plants, and when it is found that the community facilities cost more, amortization of the excess cost should be compared with the cost of servicing and maintaining the septic tank system. If the community facility is still more expensive, then the peace of mind of the property owner connected to the community system should more than make up the difference.

It should also be remembered that the cost of laying sewers in paved streets may run as much as three times more than the cost of laying them at the time that the roads and houses are built. And, finally, it is about time that the realty developers themselves began to realize that the costs of community sewerage and sewage treatment should be considered as an inherent cost of building, comparable to that of the houses to be served. It is about time, too, that more governmental officials recognized the same thing. Many officials would do well to cooperate more with developers in working out equitable and mutually advantageous arrangements for community sewerage and treatment facilities in the fringe areas. More and better planning should be done to make this possible. By far the most satisfactory method of solving fringe area sewerage problems is to connect to an existing, publicly financed sewerage system.

REFERENCES

1. Anon., "Expert Says Half of Septic Tanks in Five Midwest States Do Not Work Properly," House and Home, Apr. 1956, p. 61.
2. Kiker, John E., Jr., "New Developments in Septic Tank Systems," Proceedings of the American Society of Civil Engineers, Vol. 82, No. SA5, Oct. 1956, pp. 1088-1 - 1088-6.
3. "Manual of Septic Tank Practice," Public Health Service Publication No. 526, U. S. Government Printing Office: 1957. 85 pp. Price 35¢.
4. Stankewich, M. J., "Suburbs Need Public Sewers," Public Works, Vol. 89, No. 4, Apr. 1958, p. 154.
5. Walker, Kenneth H., "'Package' and Compact Plants—What Makes Them Tick?" Wastes Engineering.
6. Anon., "What You Need to Know About Sewage," House and Home, Feb. 1958, pp. 116-123.
7. Furman, Ralph E., "The Challenge of Metropolitan Expansion in the Development of Sewage Facilities." Proceedings of the Eleventh Annual Municipal and Sanitary Engineering Conference, Fla. Eng. & Ind. Exp. Sta. (In press).
8. Stockton, Edward, "'Packaged' Treatment Plants." Modern Sanitation, Vol. 9, No. 7, July 1957, p. 22.
9. Furman, T. deS., Kiker, J. E., and Smith, D. B., "Sewerage Planning." Florida Engineering Series No. 2, 1956, p. 63.
10. Richards, Paul W., "Sanitary Sewage Disposal in Subdivisions," Proceedings of the American Society of Civil Engineers, Vol. 83, No. SA4, Aug. 1957, pp. 1333-1 - 1333-8.
11. Kiker, John E., Jr., "Subsurface Sewage Disposal," Bulletin 23, Fla. Eng. & Ind. Exp. Sta., Gainesville, Fla., Dec. 1948, 72 pp.
12. Bradley, C. L., "The Sewage Lagoon in North Dakota," The Sanitarian, Vol. 20, No. 3, Nov.-Dec. 1957, p. 151.
13. Mitchel, Desso T., "Fringe Area Sanitation," Clean Waters for Ohio, Vol. VI, No. 2, Fall edition, 1957, pp. 11-15. (Quarterly publication of the Ohio Water Pollution Control Board, Columbus 15, Ohio.)

Paper 1715 July, 1958 SA 4

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

DESIGN DETAILS FOR INDIVIDUAL SEWAGE DISPOSAL SYSTEMS

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(Proc. Paper 1715)

SYNOPSIS

Design Procedures and simplification of design methods and comment on design of Septic Tank Systems. Some standardization is provided and methods and reasons therefor described.

INTRODUCTION

It is understood that the United States Public Health Service expects that about 450,000 Individual Sewage Disposal Systems will be built within the United States during the ensuing year. Public Health officials report that it has come to their attention that there is no unanimity of method of design of these installations and that some engineers do not want to design them on account of the small emolument that can be realized for such work and that often Boards of Health do not feel able to start definite specifications for "standard tanks" and systems designed along a rule or rules of procedure. Complete standardization would not be good if it were possible but the partial standardization of construction for individual dwellings and small businesses and industries may be rather readily accomplished.

There are considerable advantages to having a standard system of design and construction and one of these is that the construction forces become familiar with the methods of construction and so also do the inspectors. Cognizance is taken that inspectors get confused without standards and with suitable standards they are better able to direct the construction.

It is the intent of these notes and tables to furnish the design engineer or the Board of Health official with some handy and practical information on the design of the individual sewage disposal system. The data included herein are also for the purpose of simplifying and expediting the design work.

Note: Discussion open until December 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1715 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 4, July, 1958.

1. Cons. Engr., H. A. Martin and Associates, New York, N. Y.

Reasons for Design

The first question faced is, "Why design a system at all?" The answer is simple. Any suitable individual sewage disposal can only be so good as the absorbitive powers of the ground in or upon which it is to be built; it should be as near fool-proof and safe as possible; it should be large enough to accommodate the persons and their plumbing, the waste from which it is to assimilate; it should be suitable to last for many years without any repairs; and, it should otherwise be suitable to the natural conditions of the ground and other considerations such as wells, springs, streams, ground water and convenience for pumping.

Such a system should:-

a) Produce no nuisance of either odor or appearance.
b) In no way contaminate any stream or body of water.
c) Not be accessible to children.
d) Not be accessible to insects and rodents or other possible carriers.
e) Not contaminate any drinking water supply for either man or beast.

It is interesting to note herein some comments on design and the reasons therefor.

One Board of Health was most concerned because within its corporate limits there were found to be many metal tanks with tops rusted out. One masonry tank collapsed in the dead of winter with the weight of an oil truck. Baffles within some metal tanks often became destroyed or rusted away. Disposal fields often stop up because of improper construction, and breaking the pipes in the trenches.

A comment of a pumper of septic tanks was that 95% of the trouble with the individual sewage disposal system was with the disposal field or with the disposal of the clear water, whether into a disposal field, seepage pit or disposal bed. Pumping is only a palliative in troublesome cases and likely to have to be repeated in a few weeks or months.

Machine washing of clothes and dishes, lots of grease, detergents and bleaches, bad plumbing, overloading of system and long periods of wet weather are often the sources of serious trouble. Engineers can only design to take care of the probabilities and not the possibilities, so we have to make logical assumptions based on sound judgment and the information at hand.

Cesspools, as such, are not considered herein because they have a markedly limited life, produce meager treatment and are likely to require frequent pumping.

It is believed that only a masonry septic tank can meet the above generally assumed requirements, as brought out by the above comment on design. Steel tanks might be heavily coated but which might be scraped off allowing them to rust out in a few years. Precast concrete tanks have been very satisfactory if made with proper reinforcement and provided with suitably piped inlets and outlets, and the cinder block tank made, or tailored or designed to fit the conditions, although sometimes somewhat more expensive, is most likely, it is believed, to produce the best results.

It may be construed that our problem is solely or mostly with tanks. Actually it has been estimated that most of the difficulty with the individual sewage disposal is with the disposal of the comparatively clean liquid efflux from the septic tank. The ground has to be suitable and available to accept this efflux from the tank and dispose of this comparatively clear water by percolation,

transpiration, evapo-transpiration or by a combination of these.

Any design of any system such as we discuss herein must be made on the basis of the Percolation Tests made on the property by competent engineer or observer and as described herein or as per the discussion of Prof. John E. Kiker, Jr., (SA 5, Paper 1088, Vol. 82, Jour. San. Eng. Div., ASCE). Field measurements, approximations, tests, observations and opinions must be brought under the practiced eye of the trained and cautious engineer before any decision may be made.

Tests of the Ground

First the ground where the individual sewage disposal system is to be built should be tested to determine its permeability or absorbability. After reviewing thousands of the reports of these tests, it has been brought out that most of them are only approximations and some of them are not even that. It is believed that the simplest form of testing has to be adopted to get any close results.

Any such percolation tests should be made at the location and elevation at which it is intended to dispose of the clear effluent from the Septic Tank. Preferably such tests should be made between 15 and 30 inches below the depth of the finished grade and actually in the area in which leaching is to take place or the disposal trenches, leaching pit or disposal bed is to be built. The tests should be made after the test hole has been wetted down thoroughly. It is better to fill the test hole and let it leach over night, but in case only one stop is to be made, then the test should be made only after wetting the hole down as much as possible.

The percolation test should be made in an 8-inch (minimum) diameter hole. The time for the water to fall within the hole (after wetting down) should be recorded for a total of 6-inches of drop of the test water in the hole. The hole should be at least 3-ft. deep (if possible) and the test made at the elevation at which disposal within the soil is intended to take place. No percolation test should be made in freezing weather and any percolation test report should show date of test and temperature on that day.

Such tests should be made only by competent personnel (some states require that a Professional Engineer do this work), with suitable boring equipment and with an appropriate adjustable instrument or device for accurately measuring the fall of the water, such as a "Perk-Tester."

It might possibly be construed that, if an engineer laid out an individual sewage disposal system without taking a Percolation Test and giving appropriate consideration to the results of such test, that the engineer had been negligent. No system should be designed without at least one (1) percolation test.

In the making of the Percolation Tests, attention is invited to the fact that if the test is run for a 6-inch drop (as recommended herein) then the computations are simplified, for instance -

t = time per inch. T = total time for 6-inch fall. (Minutes)

$$t = \frac{T \times 60}{6} \text{ (Secs. per inch)}$$

For T = 20 Minutes:-

$$t = \frac{20 \times 60}{6} \text{ Secs.}$$

$$t = 200 \text{ Secs.}$$

$$= 3 \text{ Mins. \& } 20 \text{ Secs.}$$

It is, of course, perfectly obvious that such a calculation is one that any engineer can readily make but the simplification is included where a large number of tests are to be made, for the purpose of saving time and avoiding errors. It is to be noted that by the use of 6-inches of fall of the test water, the computations may be done and checked by inspection.

The percolation test, to be any use, has to be run at the horizontal plane at which the sewage water is supposed to be disposed or percolated. In this connection it should be borne in mind that some authorities are of the opinion that the greater amount of the water in field or bed is dissipated, not by the percolation alone, but also by transpiration. Transpiration is the absorption of the water into plant life and evaporation and probably brought about partly by the capillary action of the soil.

Because of the probable and desired transpiration of the disposable liquid from the Septic Tank, it is not good to pave the area over the disposal system. A hard standing over the area selected for the disposal might hinder if not destroy the efficiency of the disposal system, either the Bed, Disposal Field (Trenches) or the Seepage Pit.

Disposal Beds, Fields and Pits

In the area from which information is most readily obtained, it was found that the Seepage Pit is the most popular on account of the simplicity of construction of the Pit. The Disposal Field or Bed are more difficult to build and probably cost more. It is believed to be the best practice to use the Seepage Pit where practical; to resort to the Disposal Field (Disposal Trenches) in difficult cases, where the water table may be high, the soil somewhat impervious or the conditions otherwise limiting; and, it is believed that the Disposal Bed may be reserved for use in the larger installations, possibly for restaurants, hotels, motels and other large water consumers.

For any kind of commercial kitchen (restaurant, soda fountain, hotel, inn or road house), it is the best practice to have a separate line for the kitchen waste water and to run it to and through a large grease trap and thence to a Bed, Pit or Trenches of its own. The grease trap serves not only to recover the grease but to cool the effluent which may be very warm. Grease is the enemy of the Disposal Area (whether Bed, Pit or Field) and should be eliminated, if possible, in the larger installations; most authorities agree, however, that a grease trap and separate disposal is not necessary for the average domestic sewage disposal system.

Rules for Location and Design

Since an individual Disposal System has to depend on the nature of the ground for its operation, it is recommended the following rules be given serious consideration before each location is chosen and design is made:-

1. Choose location for suitable slope and drainage
2. Run a Percolation Test in the Disposal Area
3. Design system to fit the amount of effluent and the conditions of the disposal area
4. Keep the grease of kitchen waste out of the sewage system if the amounts of grease are large.

It often happens that the engineer does not have as much latitude as he would like in the selection of the location of the disposal system. It must be borne in mind that the slope of the ground must be taken into consideration, the drainage of the surface water and the elevation of the water table. Pits, Beds and Fields set below the ground water elevation will most likely give considerable trouble. Bear in mind that this ground water elevation does not stay at the same place all the time; it often rises in wet weather.

The location of the system where the surface water can run over the Tank, Trenches or Pit is almost certain to cause faulty operation. Care should be exercised to choose ground that is not subject to the overflowing of surface water because of the natural contour of the ground or from drains conducting the water to the location. Sometimes the construction of the disposal system causes the contours of the ground to be changed so much that the disposal area is placed in jeopardy from surface water infiltration. It has been reported that one owner of a large establishment had the water from roof leaders conducted to the disposal area with disastrous results.

If the percolation test is made before the grading of the land, and after the percolation test is made the grading of the land is changed more than two (2) feet, then, in that event, the percolation rate of the ground has been changed and the percolation test should be re-made to get suitable results. A disposal system set in filled ground might give serious trouble because Septic Tank, Piping and Pits or Trenches might be damaged by eventual settlement of the structures.

The vertical location (elevation) of the system must be given some thought too; it should be above the ground water, it must be low enough to accept the sewage, it must be so high in the finished grade such that access may readily be had to Septic Tank, Seepage Pit, Distribution Box and Grease Trap. Fields and Beds should not be deeper than 24-inches below the surface in order that the disposal water may be dissipated by transpiration as has been brought out herein.

Septic Tank Design Precepts

The intent of the Septic Tank is to separate or precipitate as much of the solid matter as possible and digest it. It is desirable to eliminate or precipitate all the solid particles in the sewage for digestion; the very fine particles should also be precipitated, if possible, because they stop up the Disposal Field or Bed, or the Seepage Pit. Most of the sludge may be accumulated in the first compartment of a two (2) compartment Septic Tank but a considerable amount of the smaller particles may be precipitated in the second compartment. Precipitating the smaller particles should add to the life of the Disposal System and also it probably would reduce the maintenance costs. Suitable digestion eliminates most of the solids from the effluent of the Septic Tank, in fact, much of the solid matter is turned into liquids or gases in the septic tank in the process of digestion. Some tanks have been reported to have been in service for domestic establishments for many years without any pumping, which indicates a high degree of digestion.

Frequently the engineer is faced with the design of small systems for commercial purposes, small industries, churches and offices and the usual domestic system for the two (2), three (3) or four (4) bedroom house. It appears that in the case of these smaller installations we need at least two (2) sizes of

tanks and systems; one for the commercial or small office use and the other for the small or medium sized home. It should be borne in mind that the home is where most of the sewage is likely to originate, that the sewage load is not uniform and has peaks over week-ends, etc. and mere convenience can cause an increase in domestic sewage. A four (4) bed room - four (4) bath room house occupied by the identical persons has been proven to use more water than a four (4) bed room - one (1) bath room house. It is therefore recommended that no Septic Tank for domestic use should be smaller than seven-hundred-fifty (750) gallons liquid capacity.

The small office, the commerical establishment and the drive-in or filling station might not require a very large system such that the septic tank may be of a smaller capacity. It is considered good practice to allow a five-hundred (500) gallon minimum sized septic tank for the small commercial installation. All tanks should be sized by the probable amount of water used on the premises per day.

Below are some of the consumptions of water in various kinds of occupancies which it is believed are probable:

TABLE I

SEWAGE VOLUME PER DAY PER PERSON

Type of Occupancy	Descriptive Use	Gal. per person or occupant per day.
Dwellings, Residential	Residential Clubs, Homes, Apartments	50 - 75
Food Serving Establishments, Restaurants, etc.	Sanitary Sewage	5
	Kitchen Waste	5 - 7.5
Institutional	Boarding	75 - 125
Hospitals	(Usual types)	150 - 250
Day (only)	Offices, shops, stores, industry, schools & Commercial	15 - 25
Hotels	Overnight Guests	50 - 60
Motels	" "	25

Considerable judgment should be used in determining the average daily consumption of water because some of the occupancies may be mixed. The Inn, Road House, Tavern or Hotel is a source of trouble. It is often necessary to know the number of meals served and the hours of operation. The use of this table, with discretion, will produce the proper size of septic tank and disposal field or pit in most cases.

The size of the Individual Disposal System is or may be related to the amount of water that the system is likely to have delivered to it for assimilation. No set of firm rules may be established for the use of various persons

in various contingencies, different establishments, diversified surroundings and multiplex circumstances.

It has been shown that certain plumbing fixtures are likely to produce certain flows of sewage during their periods of operation. Well within the confines of reason, there is below, a set of daily flows from sundry plumbing fixtures that has been obtained by experience and observation and estimation of the conditions likely to obtain.

Note that the following table of consumptions of water or Gals. per day of Sewage is more or less approximate and subject to the use of judgment in the application of the figures given. In the preparation of this table, liberal

TABLE II

SEWAGE FLOW FROM VARIOUS TYPE UNITS

Item	Kind & Description	Flow of Sewage therefrom Gals./Day
A	* Kitchen Sink, Domestic, Usual, Average	40 - 100
B	Clothes Washer " " "	300 - 600
C	Dish Washer " " "	200 - 400
a	Water Closet " " "	100 - 300
b	Bath Tub " " "	100 - 200
c	Shower Bath " " "	100 - 200
d	Bath Tub with shower " " "	100 - 200
e	Lavatory " " "	50 - 100
a,b,e	Combination of Tub (or shower) Water Closet and Lavatory, or complete Bath Room	250 - 600
1	Combination, Commercial Sink & Tray	300 - 900
2	Sink, Commercial	200 - 300
3	Urinal, wall lip or stall	100 - 300
4	Water Closet, Commercial, Tank	200 - 700
5	Lavatory, Commercial	100 - 300
6	Water Closet, Valve Operated	400 - 900

* Obviously the kitchen sink will use more water without a dish-washer than it will with a dish-washer, herein presume a dish-washer in the instance of the lower use of water.

reference has been made to the literature on the subject and it is believed that about the average of the various authorities' recommendations is made available herein for the design engineer. Diversification in the table has only been given limited effect, but, it is believed enough to bring out the desired features and the comparisons between different types of use. It is recommended that the use of these figures be on the liberal side where ever the conditions are unknown and the factors that influence the use of water are not apparent.

Everything in the design of the Sewage Disposal depends on the amount of water used and thus the quantity of sewage. The Septic Tank, the Grease Trap (if any), the Field, Bed or Pit have to be designed to take the maximum amount of water delivered to it and assimilate it. All of these structures are capable of variations as to size that can produce trouble in the future but the most vulnerable item is the facility for the disposal of the efflux of clear water from the Septic Tank. Careful consideration should be given to getting the Field, Bed or Pit just a bit larger than it appears to need to be. In this connection, the peak loads of the system have to be given consideration and the usual manner is to add about 50% for additional loads during periods of excessive use of water. Attention is invited to the fact that the Seepage Pit is, in essence, an underground reservoir and thus produces some provision for the peak load mentioned above. It often develops that the Seepage Pit, coupled with a Disposal Bed or trenches, produces an ideal installation for some kinds of establishments and in some kinds of situations.

Septic Tank Details

If we assume that it is better to use masonry septic tanks and that they are to be built-up of masonry and not of precast concrete, then in that event, we have to arrive at dimensions and form specifications. The cheapest masonry is likely to be cinder blocks and herein there is considered the 8-inch by 16-inch variety and it is desirable to arrange the dimensions of the septic tanks such that there would be a minimum of block cutting.

Wide tanks as compared to length are more economical to build, up to that certain width wherein additional inlets, outlets, reinforcements, etc. are, or may be, required. Any tank up to 5-feet wide needs but one inlet and one outlet, and only moderate reinforcement of the concrete top would be entailed. It is presumed that the top would be reinforced against the occasional load of a truck. This wider tank for domestic installations and a smaller one for the commercial (non-residential), (small office, filling station, store or shop) usage. This latter one need be only forty (40) inches wide. It has been found that a liquid depth of four (4) feet is ample for a septic tank and this is assumed for small tanks herein or what amounts to six (6) blocks high (of the 8" x 16" blocks, laid up in wall form). This dimension is used for all small tanks since it is determined additional depth is not necessary. Now, it is believed that we are in position to standardize economically on tanks 3-1/2 Blocks (54") and 2-1/2 Blocks (40") wide - block lengths, and 6-block (widths) high to suit by various lengths the amount of flow of the sewage; both sizes of tanks with 2-compartments for the maximum accumulation of sludge.

For the quick estimating the size and costs of septic tanks included herein are charts which give the "net Block length" and length in feet for various capacities of tanks for the two widths, namely 3-1/2 Blocks wide and 2-1/2

Blocks wide (block lengths). Refer to figures 1 and 2.

There are also similar charts for septic tanks for the larger installations, wider tanks, with multiple outlets. It is believed that the charts will save considerable time in the preparation of the standards necessary for a stipulation of tanks by a Board of Health or the design of a larger septic tank for an individual installation serving an institution or restaurant or other large user of water. Figs. 3 to 6.

It is to be noted that about a third of the volume is designed to be in the secondary compartment.

It is probable that the costs of these tanks, in general, will vary according to the number of blocks required for the various capacities. The number of blocks (approximate) for the construction of each may be taken from the charts, Figs. 1 to 6 inclusive.

EXAMPLE: Wanted - 1000 Gal. capacity 2-1/2 Blocks wide. No. of Blocks?
Net Length 7-Blocks, Primary 4-1/2 Blocks,
Secondary net length 2-1/2 Blocks (Block lengths),
and 188 Blocks, approximately are required for the
construction of this capacity tank.

Figured at $0.50 per Block "laid up," Costs, Masonry =	$102.00
" " $1.00 " " for the excavation, reinforced concrete top and bottom, and the accessories (without connecting pipe) this part of the septic tank should cost	$204.00
Total Estimated Cost of this Example	$306.00

It is contemplated that the local costs of labor and materials will be used. It is to be noted that some concrete form work is to be done and it is recommended that the lids for the hatches in the tanks be made of precast concrete. Roughly it has been found that in the Atlantic States the costs of modest sized septic tanks with connections run, on bids, between $350 and $400, the costs of the Pit, Fields, Distribution Box, etc. are additional.

The size of the Septic Tank for a specific individual installation has to be based on the use and the number of persons expected to use the system; it should be of sufficient size to fit the structure it serves and the possible or probable number of occupants. Most often the amount of domestic sewage is based on the number of bed rooms, which, of course, is actually computed upon the number of persons occupying or expected to occupy the bed rooms. No less than two (2) persons can be computed to occupy a bed room and it might be better to figure this amount to be doubled to provide for clothes and dish washers, kitchen and other uses in the average home.

In this connection it is interesting to note that one municipality has instituted application forms whereon the applicant or builder, or his engineer, is supposed to indicate the number and types of each fixture to be installed in the home or commercial installation. This has often been the source of logical information which leads to the computation of the size of the Septic Tank and the Seepage Pit or Disposal Field.

Provisions must be made for peak flow and any computations for the flow of sewage of large installations should provide tank capacity for 150% of the estimated daily flow.

$Q = 1.50\ E$ Where E is the Estimated Flow and Q is the volume used for design.

Such a method would not hold for very large installations where diversification and other engineering aspects of the system must be given consideration.

TABLE III

DOMESTIC SEPTIC TANKS

Number of Bed Rooms	1	2	3	4
Septic Tank Capacity (Gallons)	750	750	1000	1200

(It is believed that the 750-Gallon Tank should be the minimum, in order that the peak requirements may be met, as mentioned above.)

Design of Leaching Areas

Consideration must be given to the ground conditions and the nature of the soil before a decision may be made as to the size and the type of the Disposal Area. It may very well be that only a Seepage Pit may be used owing to the limited available area or because of the shape of the property and it also might be that the ground for other reasons does not permit of a large Disposal Field or a Disposal Bed. On the other hand, the Disposal Field or its modification is favored by most authorities and it is believed to be the best that can be constructed in most situations. A Bed is approximately the same thing as a Disposal Field, the essential difference is that in the Bed the entire area is excavated while in the Field only the trenches are excavated.

After the Percolation Test is received, the decision may be made as to the type of system that is to be installed. Reference to curves contained herein (Fig. 7) show the area required in square feet per gallon of flow, based on the percolation test for each of the different kinds of disposal areas, the Pit, the Field and the Bed.

Each of these systems has some advantages. The Seepage Pit is believed to be simpler to build and may be built by inexperienced workmen while the Field and Bed require careful grading and a distribution box and refinements that an ordinary unskilled workman finds difficult. The Pit furnishes a reservoir underground for the peak flow which might prove at times to be an advantage.

There are tabulations herewith which show the areas of the domestic or residential leaching system for each of 2, 3, and 4 bed room houses for various test times of the percolation test, up to 15-minutes. This table was originally made for the Inspectors of a Board of Health and has been continuously in use for years with good results. (Fig. 8)

It is to be noted that the large pits must have a reinforcement (column) in the center or otherwise the top of the larger pits will not withstand the weight of a truck which might run over the area.

In this connection it should be remembered that any driving of motor vehicles at all over the Disposal Field or the Disposal Bed is at considerable risk and also fraught with possibility of almost certain partial damage to Field or

Bed. The earth should not be packed down over Field or Bed but should be raked, seeded with grass and rolled. While it is not advisable, occasional traffic might pass over a Seepage Pit without possible damage to the operation of the Pit. Such is not recommended but the reason this is written is to bring out the fact that no traffic whatsoever may pass over Field or Bed but an occasional vehicle over the Seepage Pit would not probably hurt it. No disposal area should be paved over and no disposal area should be used as a parking lot. It is believed that the disposal area should be built in land allocated to that use only.

As has been brought out before, any disposal area should be so located or so lying that it will drain well and in such a position that no water will be conducted to it by yard drains or surface gutters and it should be so situated that the contours of the ground will not bring surface water over the area. Sometimes these requirements seem to be difficult to obtain but it is rather obvious that Fields, Beds and Pits are for the purpose of disposing of the clear sewage effluent of septic tanks and not for the disposal of surface drainage water. To presume that water conducted to a Field, Bed or Pit will not enter it and to a degree foul it would be ridiculous. Recent tests made on a large set of beds, which were so located as to receive drainage water, disclosed that after each rain they were full of water and as soon as there was clearing weather the water began to go down in the beds. If the beds or fields have to be placed in low ground which is subject to being over-run with surface water then it is necessary to catch the drainage (surface) water and conduct it in pipes or by other means over or around fields or beds and in such a manner as would prevent the drainage water from fouling the sewage disposal system. A Seepage Pit, being as it is comparatively small in area, may be more readily protected from fouling by surface drainage water.

The depth in the ground of Beds, Fields and Pits is often given all sorts of considerations. Some people who have knowledge of these things insist that the Pit or Bed be dug to such a depth as to encounter a water bearing soil. To do this in all cases may be dangerous to some one's water supply. Of course, the filtering action of the sand in the water bearing strata will have a tendency to purify the water but on the other hand it might not do it enough.

By building a Bed in such a manner as described above (to the water bearing strata) might lower the Bed to such an extent that it would be quite likely that evapo-transpiration would not take place. The evapo-transpiration is important; it works the best in the summer time when there is heat from the sun and there is growing verdure to use, absorb or transpire some of the moisture. The Disposal Trenches should be built as close to the surface as possible so as to take advantage of the evapo-transpiration.

The Disposal Pit has frequently been very successful in certain soils at a depth of four (4) feet approximately. These have been built often with just an earth bottom and dry wall sides with backfill around walls of large stone (egg size) or gravel. The reservoir action of the Disposal, Leaching, or Seepage Pit is supposed to be an advantage in that a certain amount of hydraulic head is produced on the walls and sides and this has a tendency to force the sewage water into the soil. Even though the Seepage Pit has a concrete top, there is still some evapo-transpiration through the walls and into the soil and undisturbed earth alongside the Pit. It is believed, for this reason, not good practice to pave the area over the Pit or to use this area for parking of vehicles.

It frequently develops that a home owner, a builder, or foreman or a Board of Health member has a great deal of difficulty in visualizing the relations

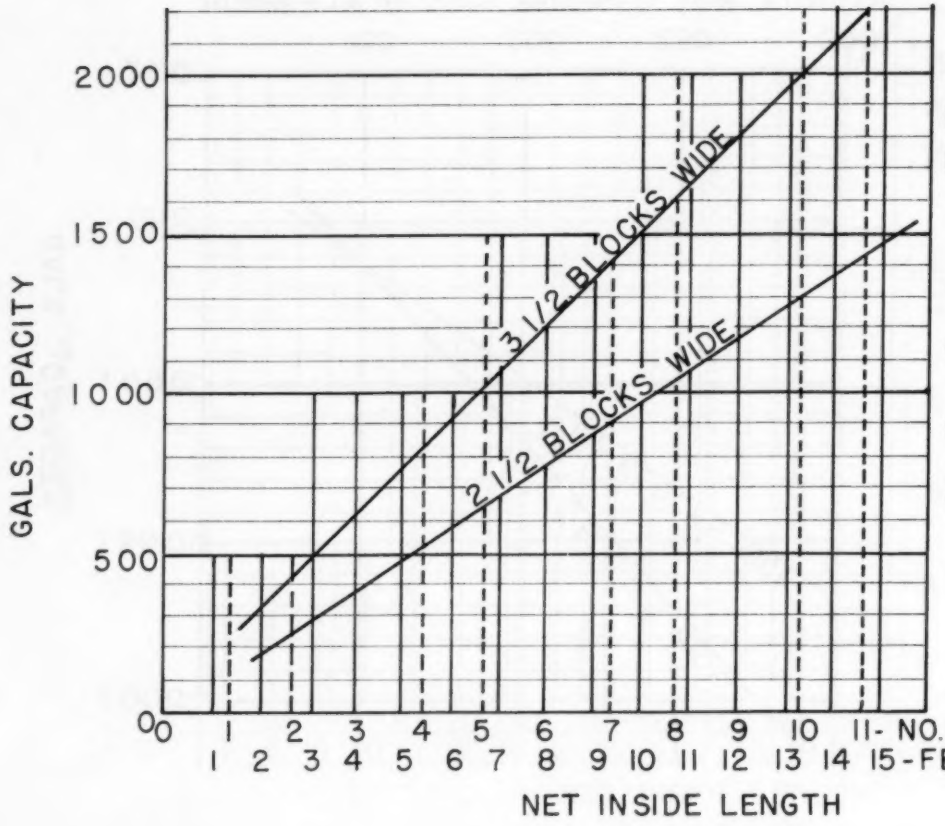
and the connections of the different structures that comprise an Individual Sewage Disposal. Sometimes it is not understood that all the pipes leading from the building to the end of the Bed or Field have to have some slope, sometimes the vertical and horizontal relative positions of the structures are not clear. For this reason a chart has been used to good advantage to instruct the uninitiated, to help the home owner or builder, to cast light on the need for the various things of which the Individual Sewage Disposal consists. Obviously such a chart is not needed by the Engineer but he can use one at an advantage at times to instruct others and, therefore, such a chart is included herein for reference. It is believed that such a chart tends to clarify some of the phases of the work that the engineer does. The chart may be of value in determining the area to be used for the Individual Sewage Disposal because it shows approximate dimensions or distances and the relative elevations. (See Figs. 9 and 10)

For the sake of clarity and for the record, it might be well to mention that the primary purpose of the Septic Tank is to precipitate and digest the solids in the sewage, and the Disposal Field, Bed or Pit are for the disposal of the clear liquid efflux from the Septic Tank. It is supposed by some that a large Septic Tank furnishes a reservoir for peak sewage load and this is true only to the degree that facilities are provided in a larger tank for the deposit of more sludge and for slower travel through the tank. Actually, if more sewage is to be received by the system over certain periods, the size of the disposal seepage or leaching area is the most important thing to consider in the design. The leaching area should be large enough to accommodate the peak flow of sewage.

SUMMARY

In order to prevent the Septic Tank System or Individual Sewage Disposal from becoming a nuisance, design methods and criteria with reasons therefor and comment thereon, are compiled herein. Tables, charts, diagrams and curves used for design and for information are included to aid in making logical decisions.

Fig. 1.


GALS. CAPACITY
2000
1500
1000
500
0
3 1/2 BLOCKS WIDE
2 1/2 BLOCKS WIDE
0 1 2 3 4 5 6 7 8 9 10 11- NO. OF BLOCK LENGTHS
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15-FEET
NET INSIDE LENGTH

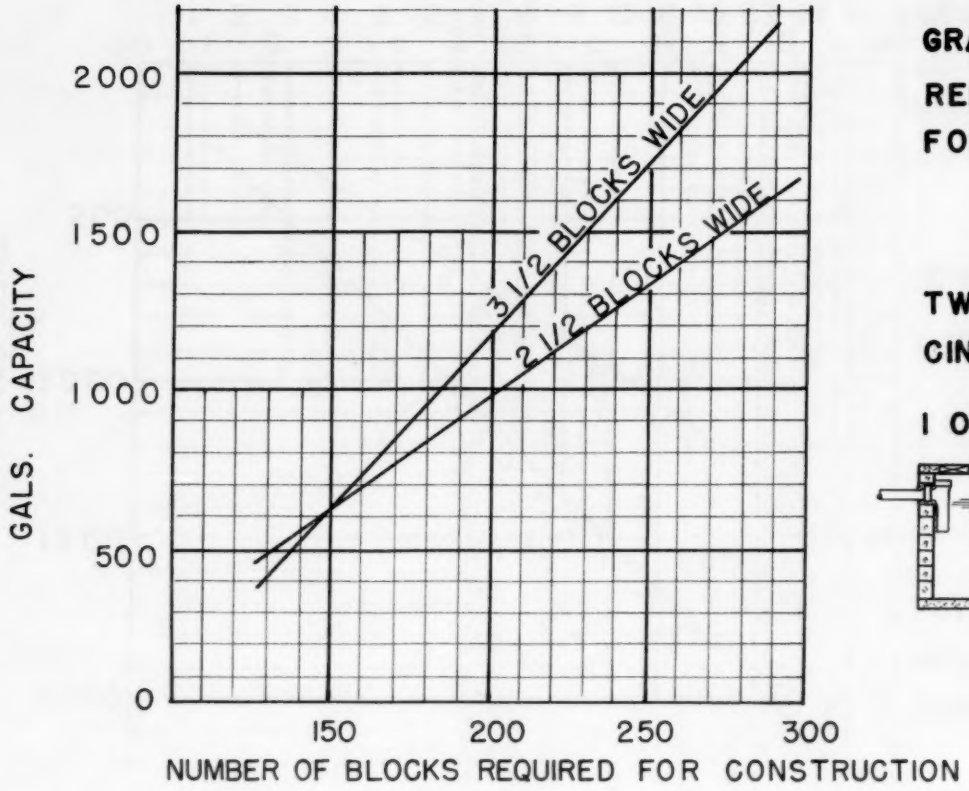
GRAPHICAL REPRESENTATION FOR DESIGN DETAILS

TWO COMPARTMENT CINDER BLOCK TANK

1 OUTLET

4'-4" LIQUID DEPTH

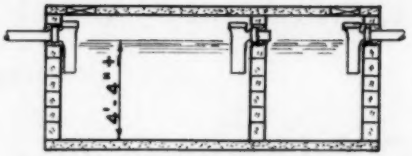
Fig.2



GRAPHICAL REPRESENTATION FOR DESIGN DETAILS

TWO COMPARTMENT CINDER BLOCK TANK

1 OUTLET



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July, 1958

Fig.3.

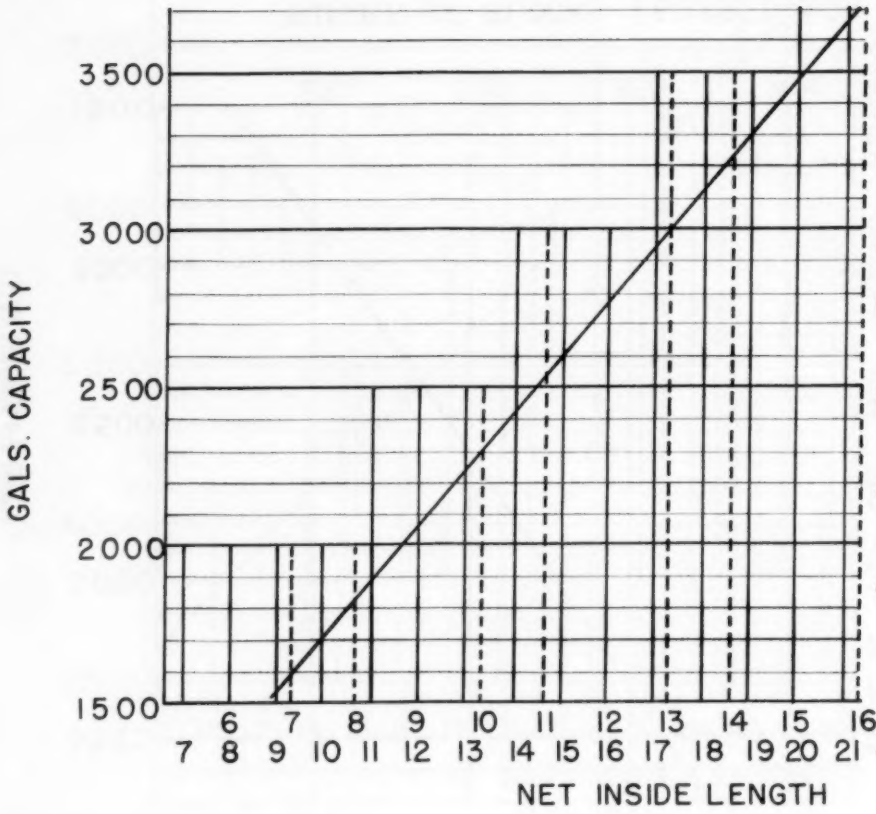
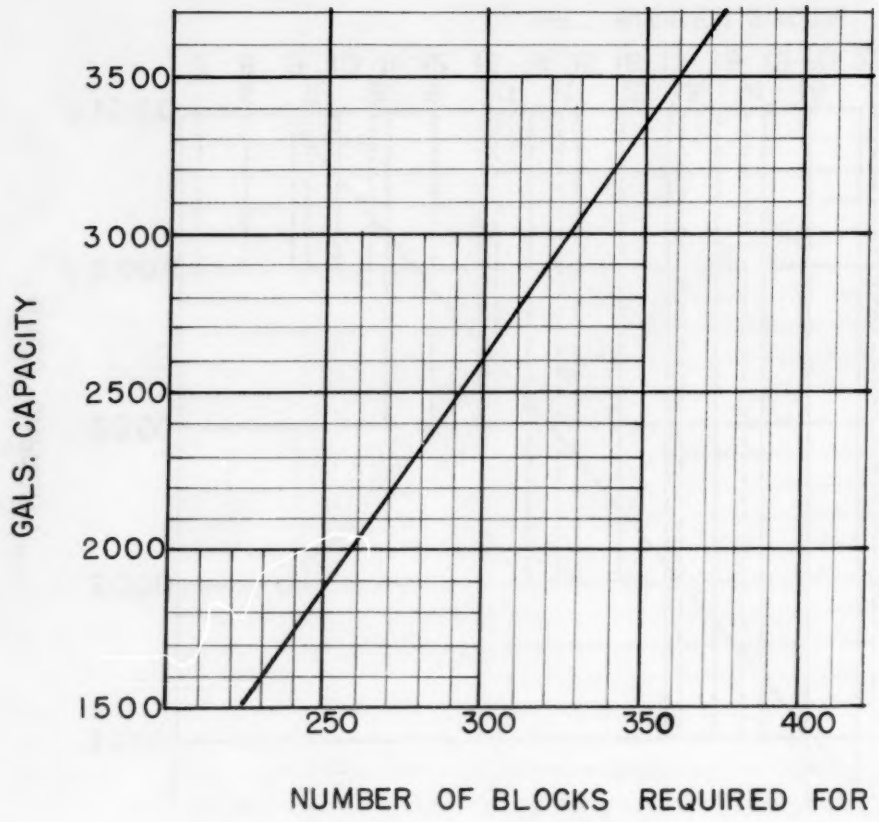

GALS. CAPACITY
3500
3000
2500
2000
1500
6 7 8 9 10 11 12 13 14 15 16 - NO. OF BLOCK LENGTHS
7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 - FEET
NET INSIDE LENGTH
GRAPHICAL
REPRESENTATION
FOR DESIGN DETAILS
TWO COMPARTMENT
CINDER BLOCK TANK
2 OUTLETS
4'-4" LIQUID DEPTH
4 BLOCKS WIDE

Fig. 4.



GRAPHICAL REPRESENTATION FOR DESIGN DETAILS

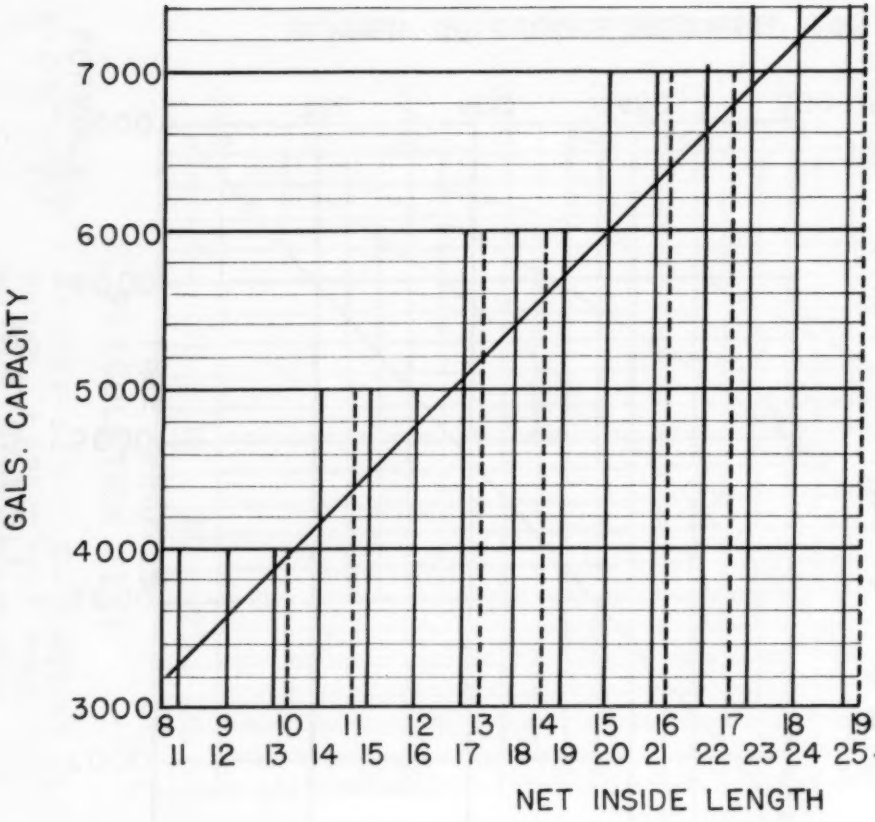
TWO COMPARTMENT CINDER BLOCK TANK

2 OUTLETS

4'-4" LIQUID DEPTH

4 BLOCKS WIDE

Fig. 5.


GALS. CAPACITY
7000
6000
5000
4000
3000
8 9 10 11 12 13 14 15 16 17 18 19 - NO. OF BLOCK LENGTHS
11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 - FEET
NET INSIDE LENGTH
GRAPHICAL REPRESENTATION FOR DESIGN DETAILS
TWO COMPARTMENT CINDER BLOCK TANK
3 OUTLETS
5' LIQUID DEPTH
6 BLOCKS WIDE

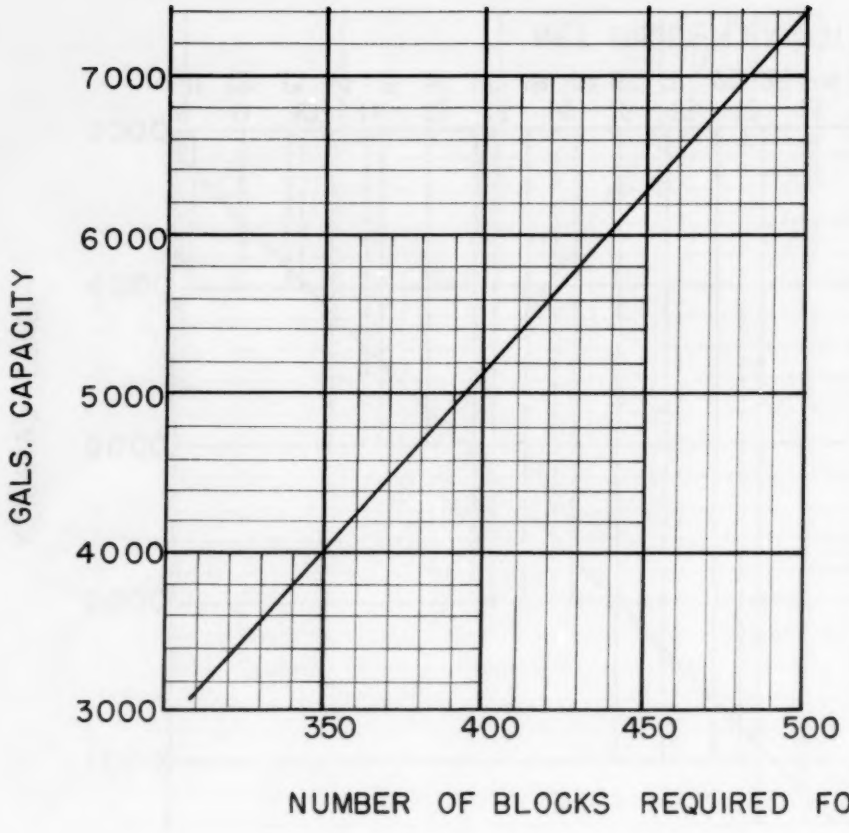


Fig. 6.

GRAPHICAL REPRESENTATION FOR DESIGN DETAILS

TWO COMPARTMENT CINDER BLOCK TANK

3 OUTLETS

5' LIQUID DEPTH

6 BLOCKS WIDE

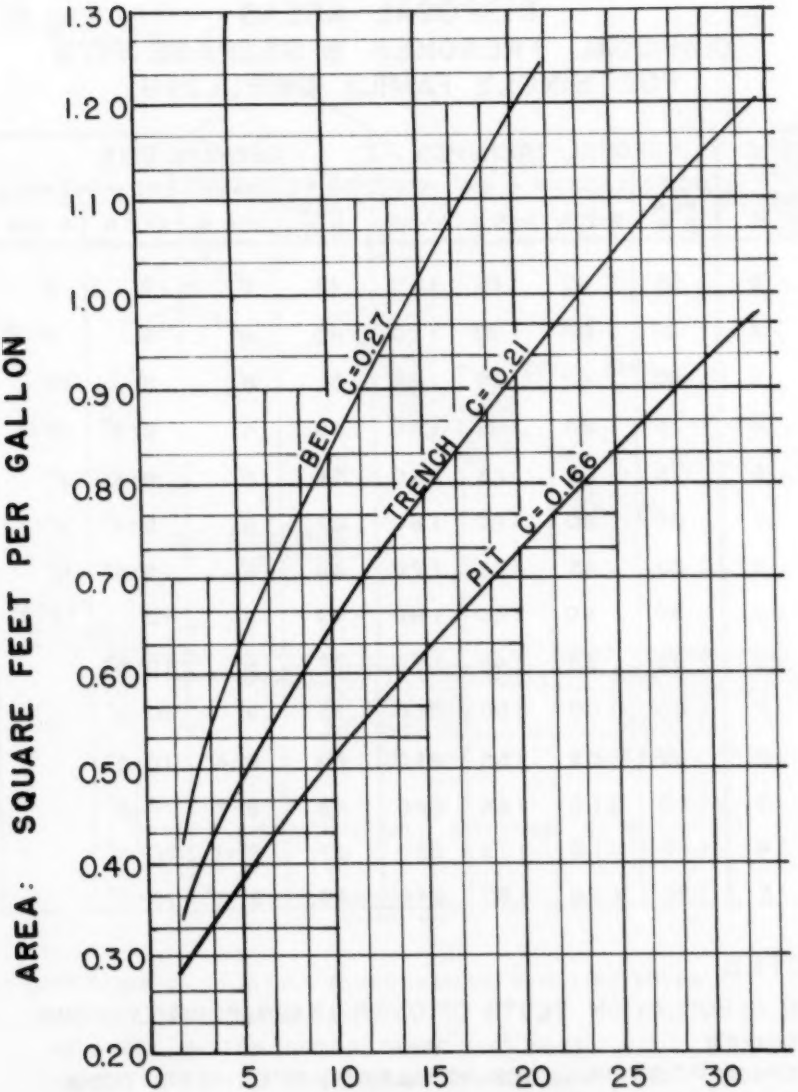


Fig. 7.

FORMULA

$$A = C\left(t^{1/2} + \frac{10}{t}\right)$$

A = AREA: SQ FT.. **C** = CONSTANT AS SHOWN. t = PERC. TEST TIME: MIN.. PIT AREA = BOTTOM & EFFECTIVE SIDE.

DISPOSAL AREAS Fig. 8.
DISPOSAL TRENCHES & SEEPAGE PITS FOR SINGLE FAMILY DWELLING

PERC. TIME MIN. PER INCH.	DISPOSAL TRENCHES				SEEPAGE PITS			
	SQ. FT. PER B.R.	LENGTH – 24" WIDE			SQ. FT. PER B.R.	DIAMETERS – OVERALL		
		2-B.R.	3-B.R.	4-B.R.		2-B.R.	3-B.R.	4-B.R.
2	55	80'	83'	110'	44	8'	8'	9'
3	60	80	90	120	48	8'	8'	9'-7"
4	65	80	98	130	51	8'	8'	10'
5	70	80	105	140	55	8'	8'-6"	10'-7"
6	75	80	113	150	58	8'	8'-10"	11'
7	80	80	120	160	62	8'	9'-4"	11'-6"
8	85	85	128	170	66	8'	9'-9"	12'
9	90	90	135	180	69	8'	10'	12'-5"
10	95	95	143	190	72	8'	10'-5"	
11	100	100	150	200	75	8'	10'-9"	
12	105	105	158	210	79	8'-4"	11'-2"	
13	110	110	165	220	83	8'-7"	11'-6"	
14	115	115	173	230	87	8'-11"	11'-11"	
15	120	120	180	240	90	9'-2"	12'-3"	

NOTES:

FOR PERCOLATION TESTS OF OVER 15 MIN. CONSULT YOUR ENGINEER.

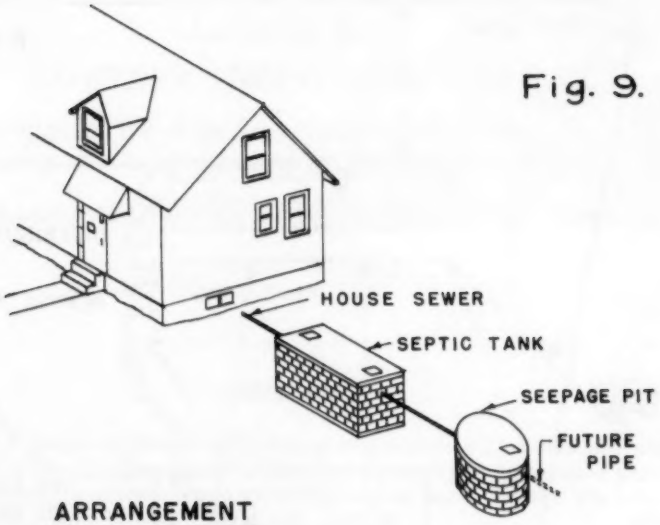
SEEPAGE PITS (OR LEACHING BASINS) OVER 12 FT. DIAM. ARE NOT RECOMENDED.

SEEPAGE PITS 9 FT. OR MORE IN DIAMETER MUST HAVE A CENTER POST TO SUPPORT THE TOP. DISPOSAL TRENCHES ARE RECOMENDED FOR THE LARGER REQUIREMENTS AND SLOW PERCOLATION AREAS.

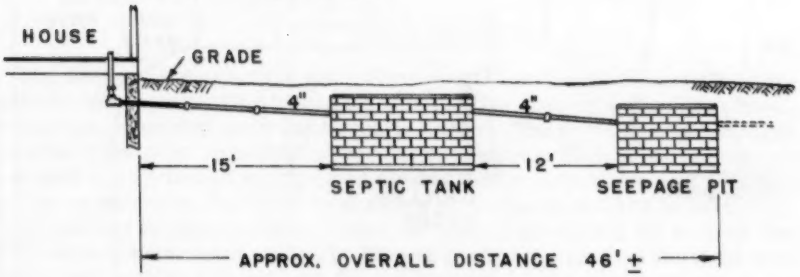
MINIMUM TRENCH LENGTH = 80 FT.

MINIMUM PIT DIAMETER = 8 FT.

Fig. 9.



ARRANGEMENT



PROFILE

SET TANKS ON SOLID (NOT FILLED) GROUND WITH DIRT COVER OF NOT MORE THAN TWO (2) FEET.

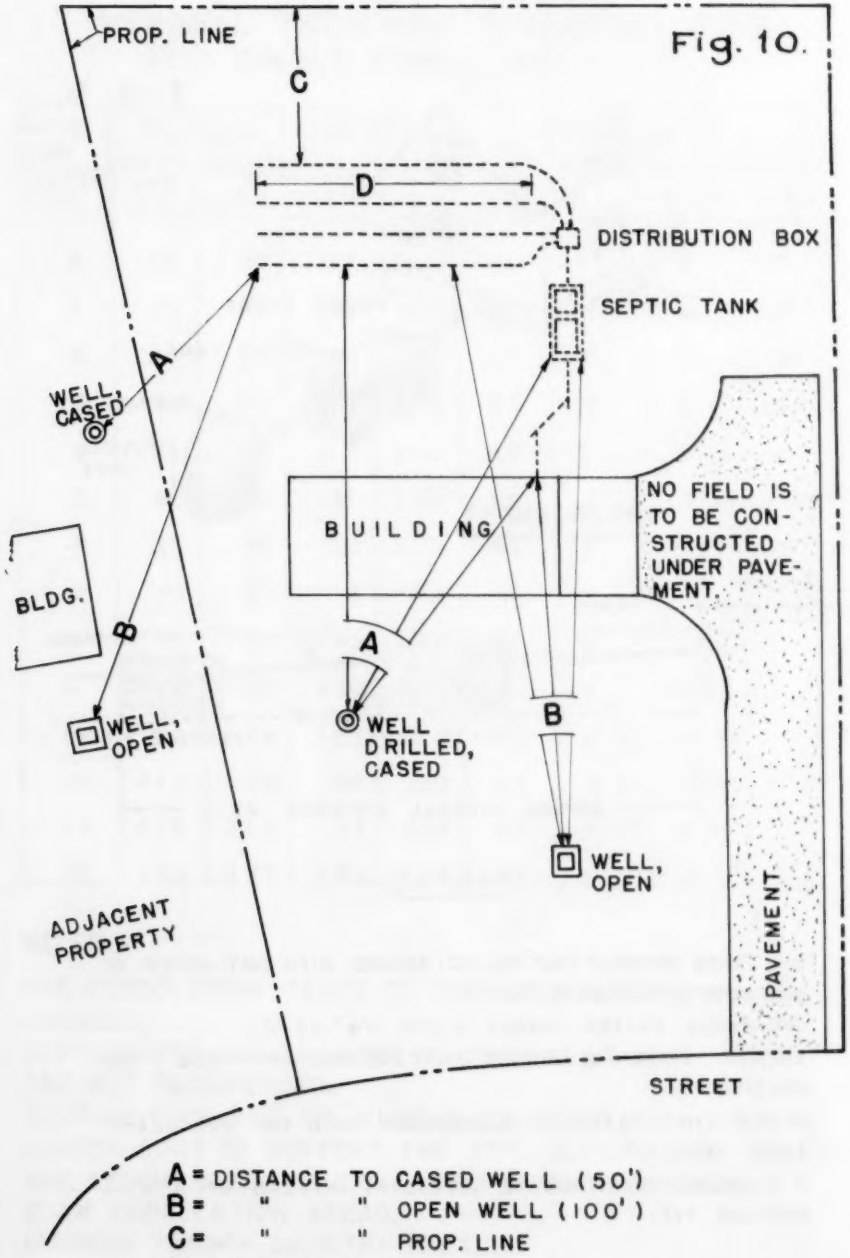
THE WHOLE SYSTEM SHOULD SLOPE 1/4" TO FT..

LOCATE SYSTEM IN GROUND THAT SLOPES AWAY FROM THE BUILDING.

DESIGN THE SYSTEM IN ACCORDANCE WITH THE PERCOLATION TEST.

A DISPOSAL FIELD MAY BE NECESSARY INSTEAD OF A SEEPAGE PIT.

Fig. 10.



A = DISTANCE TO CASED WELL (50')
B = " " OPEN WELL (100')
C = " " PROP. LINE
D = DEPENDS ON PERC. TEST

Paper 1717 July, 1958 SA 4

Journal of the
SANITARY ENGINEERING DIVISION
Proceedings of the American Society of Civil Engineers

CONTROLLED SUBMERGENCE OF PITTSBURGH'S DEEP SEWERS[a]

J. F. Laboon,[1] M. ASCE
(Proc. Paper 1717)

ABSTRACT

Features of design of Pittsburgh's thirty miles of tunnel sewers now under construction by the Allegheny County Sanitary Authority and proposed procedure for operation of the system under controlled submergence are described in this paper.

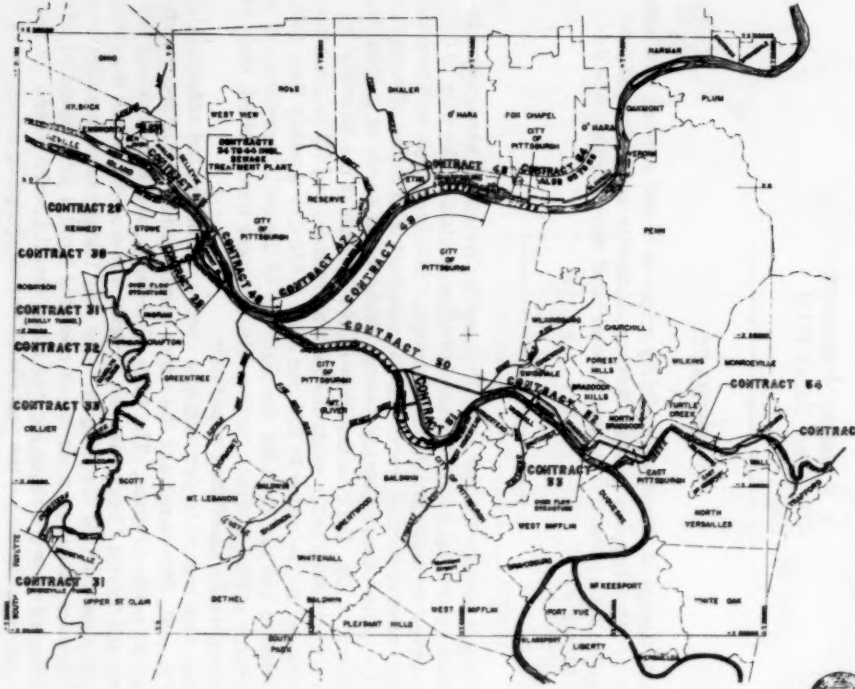
The Pittsburgh project for the collection and treatment of sewage and industrial wastes emanating from 71 municipalities, including the City of Pittsburgh, presented many problems of design. The decisions with respect to some of the most important were: (1) to construct 30.5 miles of tunnels out of a total of 69 miles of intercepting sewers designed to serve as a collecting system as shown in Fig. 1; (2) to design a major portion of the intercepting sewer system to operate under controlled submergence; and (3) to limit the initial storm water interception to 250 per cent of the dry weather flow with a further cut-back to 150 per cent for the majority of the outfall sewers and at least all the large outfalls. Since these problems were interrelated, they will be discussed together rather than separately.

Fortunately with regard to permitting a choice of design, but unfortunately with regard to cost, Pittsburgh and the other communities had no intercepting sewers along the main streams. The Ohio, Allegheny, and Monongahela Rivers and Chartiers and Turtle Creeks served as open sewers into which the outfalls from the many communities discharged. The only instances of installation of intercepting sewers, which in most cases resulted from a joint effort of adjoining municipalities, were those located along small creeks where, in the absence of acid mine drainage, the installation of intercepting sewers was

Note: Discussion open until December 1, 1958. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1717 is part of the copyrighted Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 84, No. SA 4, July, 1958.

a. Presented at meeting of ASCE, New York, N. Y., October, 1957.

1. Executive Director and Chief Engr., Allegheny County Sanitary Authority, Pittsburgh, Pa.



Note:
The coordinate system is that adopted by the U.S. Engineer Corps, Pittsburgh District.

LEGEND
Intercepting Constructed in Tunnel
Interceptor Constructed in Open Cut

Fig. 1

ALLEGHENY COUNTY SANITARY AUTHORITY
PITTSBURGH, PENNSYLVANIA

INTERCEPTING SEWERS
CONTRACTS 28 TO 64, INCLUSIVE
LOCALITY PLAN

REV.	DESCRIPTION	DATE	APP.
1			
2			
3			
4			

APPROVED

brought about as a matter of self-defense against putrefactive odors occasioned in hot weather. Thus the designers were not hampered in making a choice as to the type of the system and its controls except that tunnels had to be so located that outfall sewers could be connected to them conveniently; and the location of sewers in trench construction was limited because of the crowded banks of the streams.

At two widely separated times in the past, studies and reports were made on ways and means of collecting and treating sewage, but no action followed. The sewage and industrial wastes continued to flow untreated into the streams. It was not until 1945 when the Pure Streams Act was passed by the Pennsylvania Legislature as an amendment to a similar act in 1937 that some activity developed, resulting in the formation in 1946 of the Allegheny County Sanitary Authority which was empowered to make a study and report on a county-wide basis for a plan of collection and treatment of sewage and industrial wastes.

Although the need for collection and treatment of sewage and industrial wastes has existed for a long time in the Metropolitan Pittsburgh area, the situation was somewhat mitigated because, to quote the 1944 Federal Report of the Ohio River Committee on 'Ohio River Pollution Control', "The normal bacterial effect of untreated human wastes apparently is masked by acid pollution of the upper Ohio River". The same report stated "Additional pollution of significance includes 1,032,000 tons per year of acid wastes (calcium carbonate equivalent) which flows from the area adjacent to and above Pittsburgh."

The first report of the Sanitary Authority was made in 1948 contemplating construction of intercepting sewers in the river beds. Further studies, however, clearly indicated that such construction was inadvisable in navigable streams since the U. S. Army Engineers, who have control over such matters and from whom a permit must be obtained for such construction, would permit the construction in the river beds only with the proviso that the intercepting sewers would be subject to lowering or removal upon orders of the Army Engineers. This condition, added to the difficulty of construction, connection, control and maintenance of a river-bed intercepting sewer system, brought about a rejection of the plan and the adoption of tunnel construction in combination with trench construction where possible as being the more feasible under the circumstances.

Storm Water Interception

A review of accepted practice in large cities in regard to ratios of storm water interception to average daily flow of sanitary wastes produced values ranging from 10 to 1 to a preponderance at the ratio of 4 to 1.

Considerations which affected the selection of the ratios of storm flow interception to average daily flow of sewage and industrial wastes were (1) the availability and quantity of dilution in the streams for overflow sewage and (2) the cost. The costs were dependent upon the following factors:

(a) Size of the intercepting sewer system to carry off the intercepted storm flow, including sewage.
(b) The grades required to produce carrying velocities at storm flow to pick up the solids which may settle out due to low velocities of dry weather flow in the early years.
(c) The depth and size of pumping station and wet well.

(d) Sizes of the facilities required at the sewage treatment plant to pass the intercepted storm flows.
(e) The number, type, and effectiveness of storm water regulators.
(f) The pumping capacity required to handle the intercepted storm flow.

Consideration of all the factors involved in the determination of the capacity of the intercepting sewer system led to the following conclusions:
That,

(1) The entire system should be designed for the requirements of the year 2000 with due consideration to future development of desirable residential and industrial areas within the project area.
(2) The system should be designed only for those municipalities which have executed firm agreements with the Sanitary Authority for sewage service with no consideration to be given to providing additional capacity to serve other communities in the future.
(3) Sufficient capacity should be provided in the entire system not only for needs of sewage service in the year 2000 but also for a reasonable additional capacity for intercepted storm water.
(4) A large portion of the intercepting sewer system, including all tunnel construction, should be operated under submerged control, that is, flooded to pre-determined levels in the wet well.
(5) Settling velocities or organic matter at minimum flows during submerged operation are to be avoided.
(6) All outfall sewers of either the sanitary or combined type should be equipped with a structure for diverting to the intercepting sewer the desired flow and rejecting the excess storm flow by overflow to the rivers or creeks. Sanitary sewers are not expected to be free of roof drain connections.
(7) Capacities of intercepting sewers should be limited to an initial interception of storm water equivalent to 250 per cent of the average dry weather flow, reduced to 150 per cent of the dry weather flow after the first flush has passed.

Initial studies involved the determination of the flow from the outfall sewers and their control with regard to the quantities of storm water which would be intercepted and discharged into the intercepting sewer system. It was recognized that the number of diversion structures should be kept to a minimum because maintenance of such structures is difficult and expensive and the cost of construction is high. Such studies indicated that the minimum size of connection to the intercepting sewer system should be 8" for practical purposes and that the connection should serve as a fixed orifice. Assuming an interception of 400 per cent of average dry weather flow in 2000 A.D., and based on a design capacity of intercepting sewers of 250 per cent interception of storm water, wide area storm flow without any regulators whatsoever except for the 8" fixed orifice connections produced a hydraulic gradient with impossible conditions which was no surprise. This was rejected as impractical. A friction factor of n = .013 was used in this study although a factor as low as n = .011 will probably prevail for a long time by virtue of the high quality of concrete surface which has been produced and the type of joint which has been adopted for concrete liner pipe. Study was continued on an assumed basis of installing a limited number of controls on the large outfall sewers, such as diversion structures, with all small sewers to be constructed as 8" diameter

fixed orifice openings, hoping that the total interception would produce a satisfactory hydraulic gradient for at least the tunnel portion of the intercepting sewer system. However, it was found that with the 8" diameter orifices, of which there were many, all flows under 2 cubic feet per second dry weather flow could not be controlled within 275% dry weather flow. Even after the small sewers were assumed to be grouped by appropriate means so that one 8" connection to the intercepting sewer would serve a number of small outfall sewers, and all outfall sewers with a discharge greater than 2 cubic feet per second dry weather flow were to be controlled to average 275%, the average for the entire system was almost 350%. This was also deemed to be impractical.

In conjunction with the storm flow studies, it was necessary to give consideration to the pumping capacities which would be required to meet the various ratios of storm water interception in the year 2000 with 150, 200 and 250% of the dry weather flow being pumped out at the treatment plant. An acceptance of 275% average dry weather flow by fixed orifices or a combination of regulators and fixed orifices which would admit the same flow to the intercepting sewer, would require a pumping capacity of about 250% to maintain flow for substantially the full length of the intercepting sewers under heavy general storm conditions. Pumping a lesser capacity would have produced a condition where very little flow would have reached the intercepting sewer system at certain points upstream, with consequent discharge to the streams of the entire flow from combined sewers during storm flows. Studies of runoff and rainfall conditions indicate that flows exceeding 150% of average dry weather flow would occur for approximately 225 hours per year in 1960, and that the overflows under various conditions would reject to the river, in the case of the Monongahela River intercepting sewer system, sewage flows as follows:

Percentage of Annual Sewage Flow Rejected to Rivers

Year	Pumping 150% of Year - 2000 av. D.W. Flow Per Cent	Pumping 200% of Year - 2000 Av. D.W. Flow		Pumping 250% of Year - 2000 av. D.W. Flow Per Cent
		Fixed orifice inlets Per Cent	14 major contributors regulated Per Cent	
1945	1.70	1.65	1.55	1.55
2000	2.1	1.95	1.8	1.8

Similar but slightly different conditions would prevail in the Allegheny River intercepting sewer system.

These studies were made in 1949 on the basis of a county-wide system which was to serve 102 municipalities but, unfortunately, as far as stream pollution abatement was concerned, this project failed of adoption. Fig. 2 illustrates the sewage hydrograph anticipated in 1965 for the originally studied county-wide program. This hydrograph illustrates the possibility of the drawdown period being initiated Sunday midnight. (The average flow for the year 1970 under the adopted design is 150 mgd per day or 233 cubic feet per second). Fig. 3 shows a calculated hydrograph on the basis of an assumed condition of storm flow. According to this hydrograph, a peak of approximately 155% will reach the wet well at the main sewage pumping station.

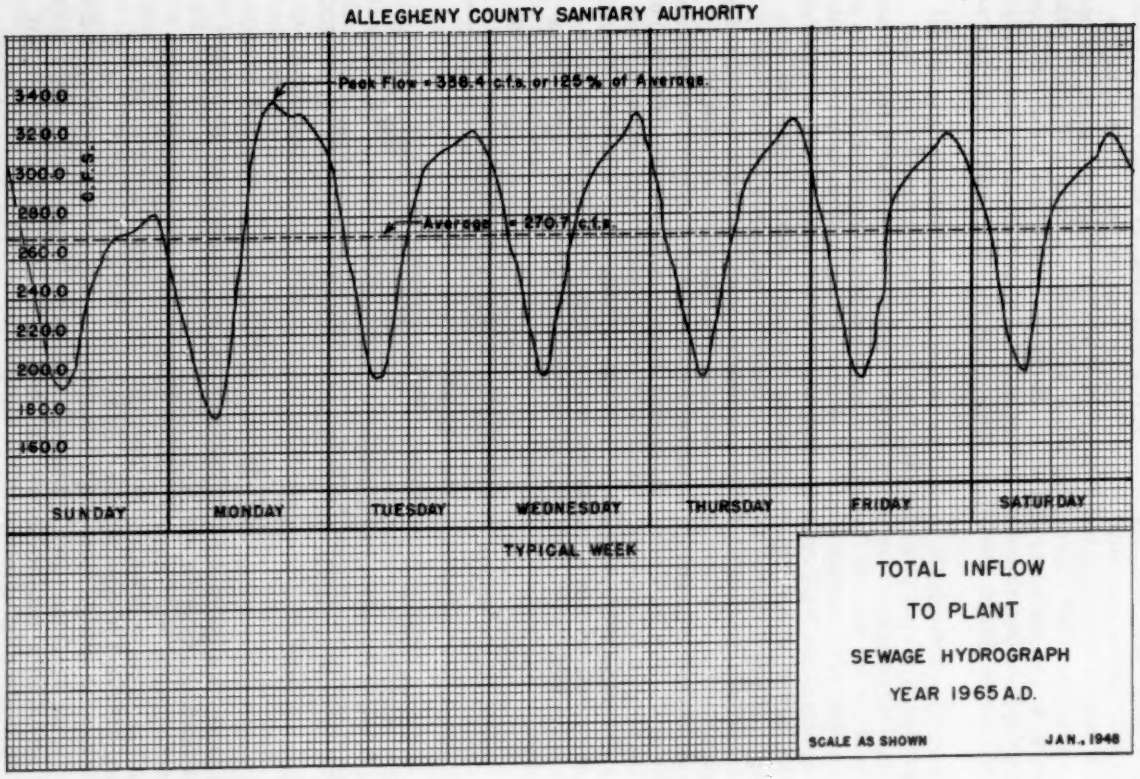

ALLEGHENY COUNTY SANITARY AUTHORITY
Peak Flow = 358.4 c.f.s. or 125% of Average
Average = 270.7 c.f.s.
340.0
320.0
300.0
280.0
260.0
240.0
220.0
200.0
180.0
160.0
SUNDAY
MONDAY
TUESDAY
WEDNESDAY
THURSDAY
FRIDAY
SATURDAY
TYPICAL WEEK
TOTAL INFLOW
TO PLANT
SEWAGE HYDROGRAPH
YEAR 1965 A.D.
SCALE AS SHOWN
JAN., 1948

Fig. 2

ALLEGHENY COUNTY SANITARY AUTHORITY

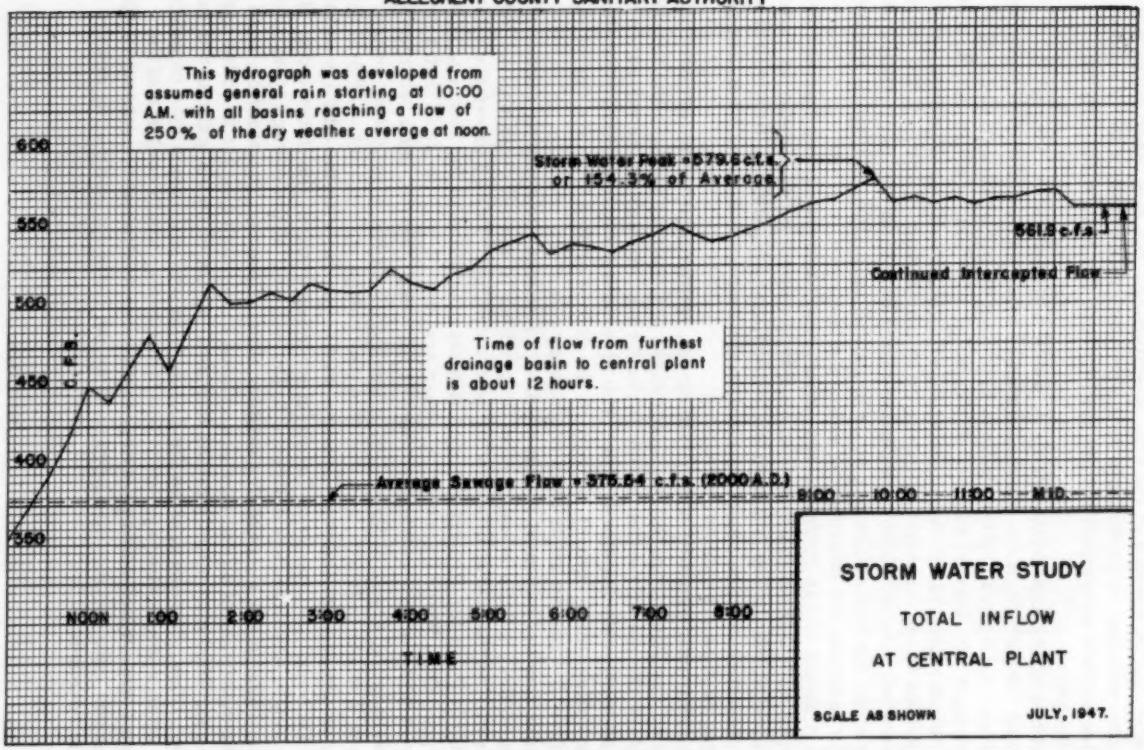

This hydrograph was developed from assumed general rain starting at 10:00 A.M. with all basins reaching a flow of 250% of the dry weather average at noon.
Storm Water Peak = 579.6 c.f.s. or 154.3% of Average
561.9 c.f.s.
Continued Intercepted Flow
Time of flow from furthest drainage basin to central plant is about 12 hours.
Average Sewage Flow = 375.54 c.f.s. (2000 A.D.)
600
550
500
450
400
350
NOON 1:00 2:00 3:00 4:00 5:00 6:00 7:00 8:00
TIME
STORM WATER STUDY
TOTAL INFLOW
AT CENTRAL PLANT
SCALE AS SHOWN
JULY, 1947.

Fig. 3

As a result of the breakdown of the county-wide program, an abbreviated program was studied and reported upon at the request of the Mayor of the City of Pittsburgh, who stipulated that this project be designed so as to produce the least cost of service to the City of Pittsburgh, but that other municipalities would be welcome to participate in the project provided the cost of rendering service to such municipalities did not increase the cost to the City of Pittsburgh.

Studies of the abbreviated project, which has been called Project Z, followed the pattern of the hydraulic studies made in 1949 for the county-wide program. As a result of the findings produced by the studies of the larger program, it was decided to place storm water controls on every outlet, thus obtaining a maximum of storm water control by permitting the storm flow to be diverted to the intercepting sewer up to a maximum of 250% of the average dry weather flow, at which time, under rising heads, an automatic tilting gate will reduce the orifice opening so as to permit only 150% of the dry weather flow to pass into the intercepting sewer system.

As a result of this decision, 151 outfall sewers of the larger type are being equipped with automatic unbalanced leaf-type control gates similar to those developed at Milwaukee and used in Chicago, and 94 double orifice type diversion structures are being installed on the small sewers. A 6" orifice plate is being used as a minimum size.

The hydraulic gradient at the 11th Street, Braddock, control and overflow chamber resulting from the pre-determined dry weather level in the wet well will place the major portion of the Turtle Creek intercepting sewer under submerged control during storm flow. The diversion structures within Turtle Creek Valley which consist entirely of the double orifice type are designed, therefore, to operate under such conditions.

Under this arrangement it was possible to reduce the pumping requirements and to produce a satisfactory hydraulic gradient to permit control of all outfalls entering the tunnel intercepting sewer system. Table I shows the sewage and storm flows for Project Z under various conditions in the Monongahela and Upper Ohio River intercepting sewer system. It will be noted that a capacity of 250% of dry weather average in the year 2000 has been provided in the intercepting sewer at the outlet of the Turtle Creek intercepting sewer, which is 7 miles in length, and that this percentage decreases to 166% at the Point, or the present Gateway Center. Furthermore, with the addition of the lower Ohio and the Chartiers Creek intercepting sewers, the latter being 14 miles long, the storm flow delivered to the wet well at the sewage treatment plant is 162% of the average dry weather flow. However, it should be noted that the quantities of storm flow are related to the high quarter of water usage in the year 2000 plus a wet weather infiltration rate of 25,000 gallons per mile per day. This, therefore, is on the liberal side and provides ample capacity in the intercepting sewer system. The lower basis, which consists of the average annual water sales plus dry weather infiltration at the rate of 19,100 gallons per mile per day, has been used as the basis for estimating sewage service revenues.

Velocities

At times during storm flow there probably will be short periods when there might be no flow in upper sections of the intercepting sewer, and short periods

TABLE I

SUMMARY OF SEWAGE QUANTITIES FOR INTERCEPTING SEWER
(Gallons Per Day)

Drainage Basin	Accumulated Year 1950 D. W. Av. Low Basis	Accumulated Year 1950 D. W. Av. High Basis	Accumulated Year 2000 D. W. Av. Low Basis	Accumulated Year 2000 D. W. Av. High Basis	Ratio, Storm Peak to Year 2000 D.W. Av. High Basis	Peak Storm Flow Year 2000
Monongahela & Upper Ohio						
Turtle Creek to 11th St., Braddock	9,494,900	11,449,800	13,981,000	20,710,000	250%	51,775,000
Braddock & N. Braddock	11,280,100	13,617,800	16,269,800	23,960,000	240%	57,504,000
Rankin & Swissvale						
Whitaker Run	13,433,000	16,258,700	19,019,200	27,415,000	230%	63,055,000
Nine Mile Run	18,153,700	22,070,500	25,354,800	35,435,000	225%	79,729,000
Homestead to Four Mile						
Four Mile Run	30,282,300	36,965,100	41,199,700	55,960,000	193%	108,003,000
Pgh. 25-26	34,547,000	42,086,100	45,798,200	62,320,000	190%	118,408,000
Pgh. 12-13	38,428,300	46,761,100	50,032,700	67,895,000	186%	126,285,000
Pgh. 27-30	38,856,100	47,282,100	50,579,100	68,595,000	185%	126,900,000
Pgh. 10-11	44,762,600	54,294,100	56,846,800	76,695,000	175%	134,216,000
Allegheny Left Bank	78,541,800	94,927,100	99,349,900	132,365,000	173%	228,991,000
Allegheny Right Bank	91,614,300	110,706,100	115,234,700	152,935,000	169%	258,460,000
Saw Mill Run	103,656,000	125,644,300	129,365,400	171,275,000	166%	284,316,000
Pgh. 19	106,794,600	129,420,300	132,633,100	175,575,000	167%	293,210,000
Pgh. 20-21	108,335,900	131,256,300	134,282,600	177,645,000	166%	294,891,000
Pgh. 22-23	110,248,400	133,592,300	136,389,300	180,365,000	166%	299,406,000
Lower Ohio	2,714,400	3,362,900	5,953,000	8,235,000	220%	18,117,000
Chartiers Creek	10,623,900	12,988,100	16,384,800	22,035,000	194%	42,827,000
TOTAL INTO PLANT	123,586,700	149,943,300	158,727,100	210,635,000	162%	340,807,000 *

Note: Low Basis means based on annual water sales, infiltration at 19,100 gal. per mile per day, and probable population.

High Basis means based on high quarter water sales, infiltration at 25,000 gal. per mile per day, and interceptor design population.

* This figure is reduced from the total of the Upper Ohio, Lower Ohio and Chartiers Creek peaks because the peaks from these three intercepting sewers do not reach the wet well simultaneously.

when there might be flow at very low velocities during the transition period when the rate of runoff diminishes or decreases. Since these periods of less than cleansing velocities would normally be of only a few hours duration, it is not felt that deposits in the intercepting sewer would be likely to seriously resist normal minimum scouring velocities. It may also be noted that during the period of storm overflow, while there is no flow in the upstream section of the intercepting sewer under consideration, all heavy storm flows above that section carrying the maximum amounts of heavy solids would be rejected to the river, so little of these potential deposits would be carried into the intercepting sewer.

Since there is a possibility that during low dry weather flows some solids may be deposited due to low velocities in the portion of the intercepting sewer system operating under submerged control, it is proposed to accelerate the flow in the intercepting sewer system to the highest possible cleansing velocity by pumping down the wet well and the intercepting sewer system on a weekly basis so as to obtain a free flow condition. This would be done during a period of minimum weekly flow which our studies indicate would occur between midnight Sunday and noon Monday. Table II shows the velocities under various conditions of flow in the Monongahela and Upper Ohio intercepting sewer system under submerged control. It may be deduced that storm flow in 1959, when the system will go into operation, will produce minimum velocities close to 2 feet per second which may occur as often as 60 times a year. This would supplement the drawdown periods of cleansing velocities. Table III shows the velocities for the same system during a drawdown period and Table IV shows the depths of sewage in the intercepting sewers at various stations during drawdown.

Under submerged flow conditions in the Monongahela River intercepting sewer system, including that portion of the main intercepting sewer from the Point to the treatment plant and the Upper Ohio River, the velocities at 60% of the average estimated flow for the year 1950 were calculated to be as low as .58 feet per second. This value becomes 1 foot per second in 2000. However, at 150% of the average daily flow estimated for the year 1950, the minimum velocity in the system would be 1.45 feet per second. While the low velocities may produce some settling out of solids, the weekly drawdown for the same system in 1950 was estimated to produce velocities ranging from 3.1 to 4.4 feet per second during the hours of lowest flow. Since the drawdown is estimated to require an operation of 8 hours under 1970 conditions, allowing 4 hours for pumpdown and 4 hours for operation at pumpdown conditions, the average velocity will be somewhat greater than the minimum and should cleanse the system of all deposits.

The depths of flow during drawdown conditions for the years 1950 and 2000 in the Monongahela and Upper Ohio system are illustrated in Fig. 4 at different points in the system under various conditions. It will be noted that the difference in depth of flow in the various parts of the system between the lowest calculated flow for the year 1950 and the highest calculated flow for the year 2000 is less than 2 feet, thus indicating almost constant hydraulic conditions except for velocities.

The location of the intercepting sewer system as finally designed is shown in Fig. 1. The main sewers in tunnel, all of which will be under submerged control, are set apart from the rest of the system by dot and dash lines. The two main overflow stations controlling long lines of intercepting sewer are indicated by circles, one at 11th Street, Braddock for control of flows from

TABLE II
MONONGAHELA AND OHIO RIVERS

VELOCITIES IN INTERCEPTING SEWER FROM ELEVENTH STREET, BRADDOCK, TO THE SEWAGE TREATMENT PLANT
INTERCEPTING SEWER FLOWING FULL DURING SUBMERGED CONDITION

Portion of Intercepting Sewer		Diameter	Area Sq. Ft.	Year 1950 Ave. D. W. Flow Q c.f.s.	Velocities in Ft. per Sec. 60% Q	100% Q	150% Q	Year 2000 Ave. D.W. Flow Q c.f.s.	Velocities in Ft. per Sec. 60% Q	100% Q	150% Q	Year 2000 Storm Flow Q c.f.s.	Velocity Ft. per Sec.
11th Street, Braddock	Upper End	54"	15.90	15.36	.58	.97	1.45	26.54	1.00	1.67	2.50	66.42	4.18
to Whitaker Run Connection	Lower End			19.22	.72	1.21	1.81	31.92	1.20	2.01	3.01	74.93	4.71
Whitaker Run Connection	Upper End	60"	19.64	20.88	.64	1.06	1.60	34.39	1.05	1.75	2.63	79.10	4.03
to Nine Mile Run	Lower End			22.19	.68	1.13	1.70	35.98	1.10	1.83	2.75	82.76	4.21
Nine Mile Run	Upper End	66"	23.76	31.18	.79	1.31	1.97	48.39	1.22	2.04	3.06	108.88	4.58
to Four Mile Run	Lower End			31.18	.79	1.31	1.97	48.39	1.22	2.04	3.06	108.88	4.58
Four Mile Run	Upper End	78"	33.18	55.03	1.00	1.66	2.49	81.27	1.47	2.45	3.67	156.86	4.73
to So. Twenty-Second Street	Lower End			60.51	1.09	1.82	2.74	88.01	1.59	2.65	3.98	167.22	5.04
So. Twenty-Second Street	Upper End	84"	38.48	65.10	1.02	1.69	2.54	93.51	1.46	2.43	3.64	177.42	4.61
to So. Eighth Street	Lower End			67.82	1.06	1.76	2.64	96.77	1.51	2.52	3.77	183.87	4.78
So. Eighth Street	Upper End	90"	44.18	70.19	.95	1.59	2.38	99.70	1.35	2.26	3.39	184.45	4.17
to Point	Lower End			81.04	1.10	1.83	2.75	112.23	1.52	2.54	3.81	195.53	4.42
Point River Crossing		114"	70.88	145.58	1.23	2.06	3.09	200.52	1.70	2.83	4.24	346.91	4.89
Point River Crossing	Upper End	120"	78.54	169.40	1.29	2.15	3.24	231.57	1.77	2.95	4.42	391.36	4.98
to Belmont Street	Lower End			169.99	1.30	2.16	3.25	232.35	1.78	2.96	4.44	385.70	4.91
Belmont Street	Upper End	126"	86.59	192.99	1.34	2.23	3.34	261.28	1.81	3.02	4.53	433.72	5.01
to Treatment Plant	Lower End			204.81	1.42	2.37	3.55	274.79	1.90	3.17	4.76	456.15	5.27

TABLE III

MONONGAHELA AND OHIO RIVERS

VELOCITIES IN INTERCEPTING SEWER FROM ELEVENTH STREET, BRADDOCK, TO THE SEWAGE TREATMENT PLANT
INTERCEPTING SEWER FLOWING PARTIALLY FULL DURING DRAWDOWN CONDITION

Portion of Intercepting Sewer		Invert Elev.	Diameter	Area Sq. Ft.	Slope	Year 1950 Ave. D.W. Flow Q c.f.s.	Velocities in Ft. per Sec. 50% Q	100% Q	150% Q	Year 2000 Ave. D.W. Flow Q c.f.s.	Velocities in Ft. per Sec. 50% Q	100% Q	150% Q	Year 2000 Storm Flow Q c.f.s.	Velocity Ft. per Sec.
11th Street, Braddock to	Upper End	681.50	54"	15.90	.0014	15.36	3.1	3.7	4.1	26.54	3.7	4.3	4.8	66.42	5.4
							(3.3)	(3.9)	(4.4)		(3.9)	(4.5)	(5.0)		
Whitaker Run Connection	Lower End	666.79				19.22	3.4	3.9	4.4	31.92	4.0	4.5	5.0	74.93	5.5
Whitaker Run Connection to	Upper End	666.79	60"	19.64	.00136	20.88	3.4	3.9	4.4	34.39	3.9	4.5	5.1	79.10	5.6
								(3.9)	(4.2)			(4.4)	(4.7)		(5.1)
Nine Mile Run	Lower End	657.01				22.19	3.4	4.0	4.5	35.98	4.0	4.6	5.1	82.76	5.6
Nine Mile Run to	Upper End	656.76	66"	23.76	.00123	31.18	3.6	4.2	4.8	48.39	4.1	4.8	5.3	108.88	5.8
									(4.3)			(4.5)	(4.5)		(5.3)
Four Mile Run Shaft	Lower End	643.63				31.18	3.6	4.2	4.8	48.39	4.1	4.8	5.3	108.88	5.8
Four Mile Run Shaft to	Upper End	642.96	78"	33.18	.000977	55.03	3.9	4.5	5.0	81.27	4.3	5.0	5.5	156.86	5.7
								(4.5)	(4.9)		(4.4)	(4.8)	(5.3)		(5.5)
So. Twenty-Second Street	Lower End	635.74				60.51	4.0	4.6	5.1	88.01	4.4	5.1	5.6	167.22	5.7
So. Twenty-Second Street to	Upper End	635.57	84"	38.48	.000716	65.10	3.6	4.2	4.7	93.51	4.0	4.6	5.1	177.42	5.2
								(4.2)	(4.5)		(4.0)	(4.5)	(4.9)		
So. Eighth Street	Lower End	632.18				67.82	3.7	4.2	4.7	96.77	4.1	4.7	5.1	183.87	5.2
So. Eighth Street to	Upper End	632.01	90"	44.18	.000544	70.19	3.3	3.9	4.3	99.70	3.7	4.3	4.6	184.45	4.7
								(4.0)	(4.2)			(4.1)	(4.4)		
Point	Lower End	628.19				81.04	3.5	4.0	4.5	112.23	3.8	4.4	4.7	195.53	4.6
Point River Crossing	Upper End	627.44	114"	70.88	.00062	145.58	4.3	4.9	5.4	200.52	4.7	5.3	5.8	346.91	5.9
							(3.8)	(4.3)	(4.8)		(4.2)	(4.7)	(5.2)		(5.5)
	Lower End	626.63				145.58	4.3	4.9	5.4	200.52	4.7	5.3	5.8	346.91	5.9
Point River Crossing to	Upper End	626.63	120"	78.54	.00058	169.40	4.2	4.9	5.4	231.57	4.6	5.3	5.8	391.36	5.8
							(3.9)	(4.5)	(5.0)		(4.3)	(5.0)	(5.4)		(5.6)
Belmont Street	Lower End	625.07				169.99	4.2	4.9	5.4	232.35	4.6	5.3	5.8	385.70	5.8
Belmont Street to	Upper End	625.07	126"	86.59	.00057	192.99	4.4	5.0	5.6	261.28	4.8	5.5	5.9	433.72	6.0
Treatment Plant	Lower End	618.25				204.81	4.4	5.2	5.7	274.79	4.8	5.5	6.0	456.15	6.0

NOTE: Figures in parentheses show velocities caused by backwater from section immediately downstream.

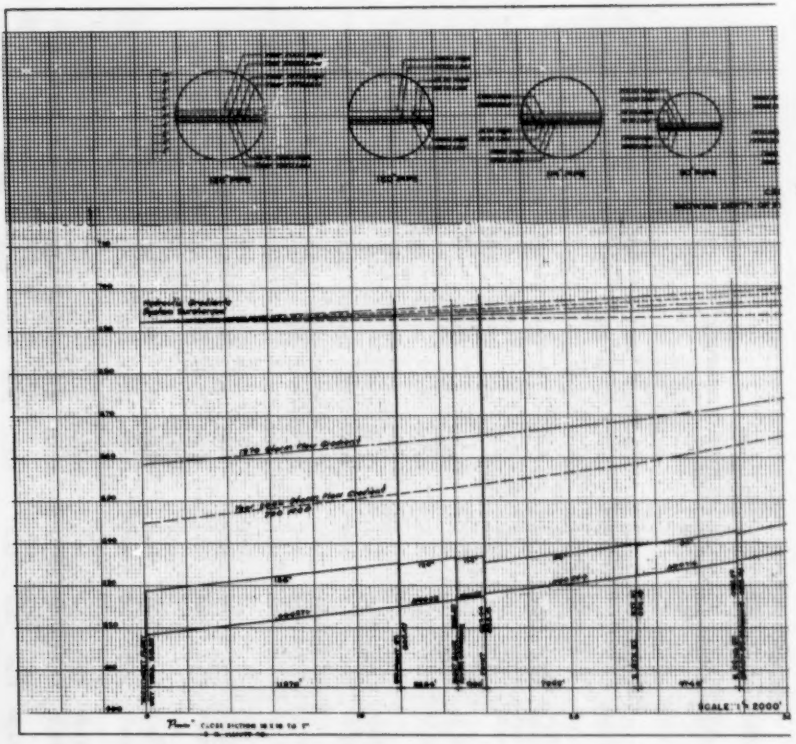
TABLE IV

MONONGAHELA AND OHIO RIVERS

DEPTHS OF FLOW IN INTERCEPTING SEWER FROM ELEVENTH STREET, BRADDOCK, TO THE SEWAGE TREATMENT PLANT
INTERCEPTING SEWER FLOWING PARTIALLY FULL DURING DRAWDOWN CONDITION

Portion of Intercepting Sewer		Invert Elev.	Diameter	Area Sq. Ft.	Slope	Year 1950 Ave. D. W. Flow Q c.f.s.	Depths of Flow in Inches 60% Q	100% Q	150% Q	Year 2000 Ave. D. W. Flow Q c.f.s.	Depths of Flow in Inches 60% Q	100% Q	150% Q	Year 2000 Storm Flow Q c.f.s.	Depth of Flow in Inches
11th Street, Braddock to	Upper End	681.50	54"	15.90	.0014	15.36	12.7	16.3	20.1	26.54	16.6	21.7	27.3	66.42	38.2
							(14.8)	(18.9)	(23.3)		(18.8)	(24.5)	(30.9)		
Whitaker Run Connection	Lower End	666.79				19.22	14.2	18.4	22.1	31.92	18.3	23.9	30.5	74.93	41.3
Whitaker Run Connection to	Upper End	666.79	60"	19.64	.00136	20.88	14.8	18.9	23.3	34.39	18.8	24.5	30.9	79.10	40.7
								(20.1)	(25.6)			(26.2)	(34.0)		(46.2)
Nine Mile Run	Lower End	657.01				22.19	15.1	19.5	24.1	35.98	19.3	25.2	31.7	82.76	42.2
Nine Mile Run to	Upper End	656.76	66"	23.76	.00123	31.18	17.8	23.1	28.6	48.39	22.2	29.2	37.0	108.88	49.2
									(31.1)			(30.7)	(41.8)		(52.9)
Four Mile Run Shaft	Lower End	643.63				31.18	17.8	23.1	28.6	48.39	22.2	29.2	37.0	108.88	49.2
Four Mile Run Shaft to	Upper End	642.96	78"	33.18	.000977	55.03	23.9	31.1	39.1	81.27	29.3	38.7	49.8	156.86	60.9
								(33.4)	(42.7)		(30.7)	(41.9)	(54.4)		(67.4)
So. Twenty-Second Street	Lower End	635.74				60.51	25.1	32.9	41.4	88.01	30.4	40.6	52.8	167.22	65.4
So. Twenty-Second Street to	Upper End	635.57	84"	38.48	.000716	65.10	27.0	35.4	44.7	93.51	32.7	43.9	56.4	177.42	69.4
								(36.9)	(47.0)		(33.6)	(45.4)	(59.8)		
So. Eighth Street	Lower End	632.18				67.82	27.6	36.3	45.8	96.77	33.3	44.4	58.0	183.87	72.7
So. Eighth Street to	Upper End	632.01	90"	44.18	.000544	70.19	29.6	38.9	49.0	99.70	35.6	47.4	61.8	184.45	74.7
Point								(42.3)	(56.0)			(52.6)	(72.5)		
	Lower End	628.19				81.04	31.9	42.0	53.6	112.23	38.0	51.0	68.2	195.53	84.2
Point River Crossing	Upper End	627.44	114"	70.88	.00062	145.58	38.8	51.3	65.0	200.52	46.2	61.6	81.5	346.91	92.9
							(41.4)	(54.6)	(69.0)		(48.8)	(65.4)	(86.4)		(95.4)
	Lower End	626.63				145.58	38.8	51.3	65.0	200.52	46.2	61.6	81.5	346.91	92.9
Point River Crossing to	Upper End	626.63	120"	78.54	.00058	169.40	41.4	54.6	69.0	231.57	48.8	65.4	86.4	391.36	95.4
							(44.1)	(58.0)	(73.7)		(51.7)	(68.7)	(90.7)		(98.9)
Belmont Street	Lower End	625.07				169.99	41.4	54.6	69.0	232.35	48.8	65.4	86.4	385.70	95.2
Belmont Street to	Upper End	625.07	126"	86.59	.00057	192.99	44.1	58.0	73.7	261.28	51.7	68.7	90.7	433.72	98.9
Treatment Plant	Lower End	618.25				204.81	45.4	61.1	76.2	274.79	52.9	70.6	94.5	456.15	104.6

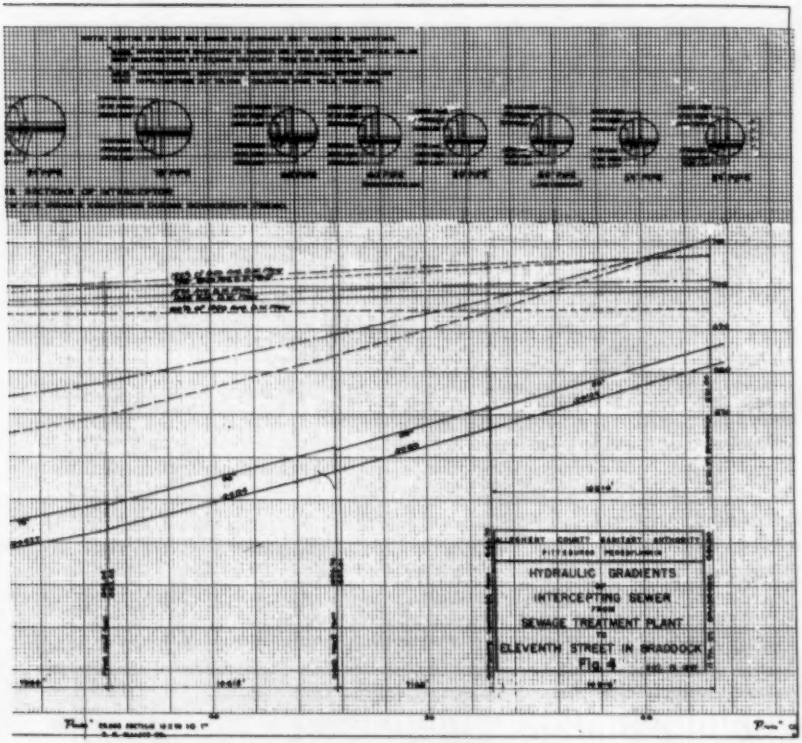
NOTE: Figures in parentheses show depths of flow caused by backwater from section immediately downstream.



approximately 8 miles of the Turtle Creek and Braddock intercepting sewer, and the other at the mouth of Chartiers Creek to control nearly 14 miles of intercepting sewers.

The overflow chamber for the Turtle Creek system consists of an adjustable sluice gate located at the bottom of the chamber with an adjustable overflow weir near the top of the chamber. The sluice gate at the bottom will be maintained partially open to permit constant flow-through conditions, allowing grit to pass through into the Monongahela River intercepting sewer. Approximately 1/3 of the flow entering the Monongahela intercepting sewer will be carried through the lower orifice formed by the sluice gate. The remainder of the dry weather flow will pass through two orifices near the top of the chamber at elevation 711.5 at about the calculated hydraulic gradient of the Monongahela River intercepting sewer. The overflow weir is to be located at elevation 712.25. Fig. 5 illustrates the flows and overflows under varying hydraulic conditions in the system at this point. It will be seen that the flow from the Turtle Creek intercepting sewer and in fact from many other connections, can be readily controlled and increased or reduced by lowering or raising the level of the flow line in the wet well at the pumping station to produce the desired hydraulic gradient under various conditions of flow. This is a very desirable characteristic of submerged control.

The hydraulic gradient based on the present design of Project Z is illustrated in Fig. 4. A hydraulic gradient, assuming a normal operating level under submerged control of elevation 692 in the wet well, produces a gradient under



design flows such as to satisfy the requirements of submerged control at the upper ends of the tunnel system. Hydraulic gradients are shown for storm flow conditions in the years 1970 and 2000; also for minimum flow based on 1950 quantities and for average flows in the years 1950, 1970 and 2000. The anticipated average daily flow for the year 2000 is 185 mgd. The hydraulic gradient for 150% of the 1950 average dry weather flow is very little different from that for the average dry weather flow anticipated for the year 2000. The design sizes and grades of the intercepting sewer system for the Monongahela Valley and Upper Ohio system are shown in Fig. 4 together with depths of flow at various points under varying conditions already referred to in this paper.

Telemetering

Data on the behavior of the hydraulic gradient at strategic points resulting from submerged control under all conditions will be supplied by means of eleven indicating telemeters with indicators to be grouped on a master control board located in the control room of the pumping station. By means of such data, local storms may be located as they affect the intercepting sewer system at a particular point, and preparations may then be made for drawing down the wet well and thus increasing the grade of the hydraulic gradient to care for the increased flow to the wet well. The telemeters will also serve as a guide to normal operation of the intercepting sewer system.

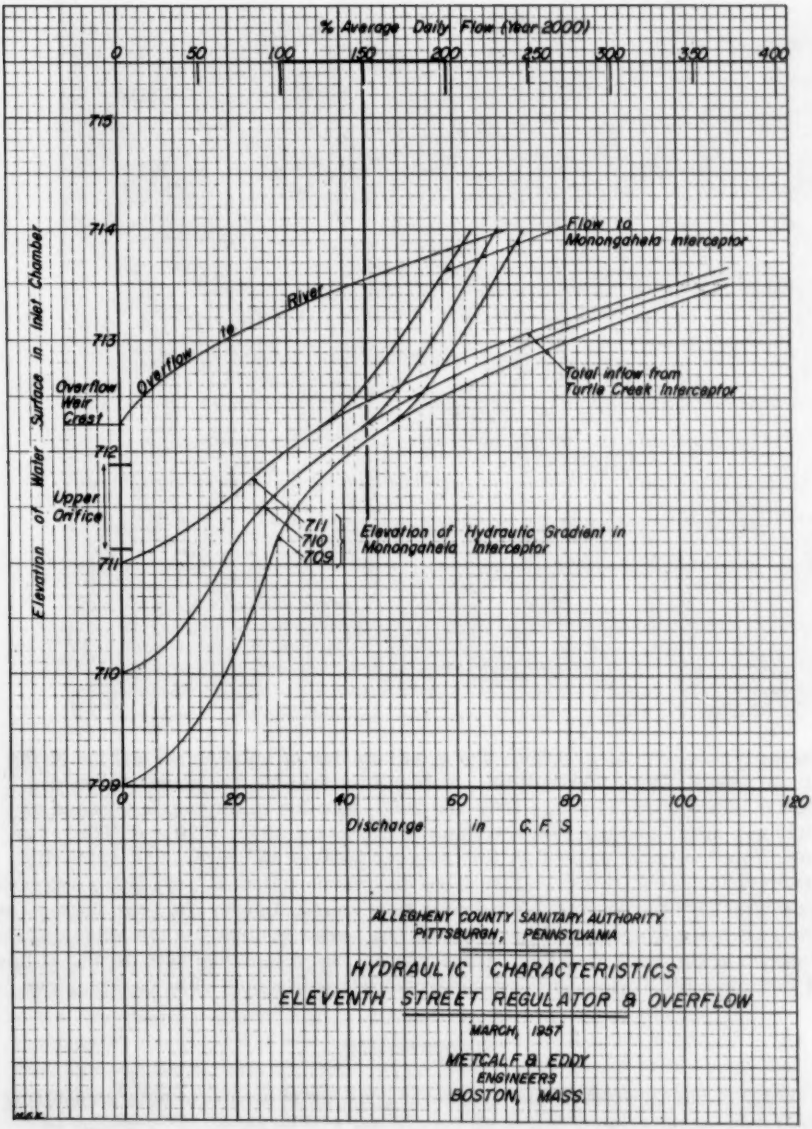


Fig. 5

32921-Z-11684

The overflow chamber at the lower end of the Chartiers Creek intercepting sewer system will be equipped with a recording flow meter, since this intercepting sewer is the most important part of the system aside from the intercepting sewers along the rivers.

Pumping Rates and Requirements at Treatment Plant

The effectiveness of the storm water overflow devices, the type of operation and the carrying capacity of the intercepting sewer system have a direct bearing on the type, size, and number of pumping units which will be necessary to meet the varying flow conditions. The head conditions at the pumping station vary greatly inasmuch as normal pumping head under submerged control may be as little as 39 feet, whereas under drawdown conditions, the pumping head may be as great as 106 feet. It is most economical to have pumping units that can provide the desired capacities at all operating levels but this sometimes is impractical. Additional pumps often are required to provide the necessary capacities at radically different dynamic heads. With storm flow, the anticipated drawdown to provide for increased flows will produce a calculated dynamic head of 78 feet.

To provide sufficient capacity at the various design heads, and to meet the conditions of design with respect to storm flow interception, 5 pumps are being installed, 3 designed for 2-speed operation, and 2 with variable speeds. The rated capacity of each of the 5 pumps is 60 mgd, but the capacities vary greatly under the varying operating heads.

During weekly-drawdown, anticipating an average dry weather flow of 150 mgd in the year 1970, and a minimum flow at 60%, or a rate of 90 mgd, 5 pumps will produce a capacity of 200 mgd or 133% of the design capacity of the sewage treatment plant. This will be sufficient for drawdown plus normal sewage flow during such periods.

For storm flow the 5 pumps will produce a total capacity of 322 mgd, or 215% of the 1970 dry weather flow. This capacity is more than ample to meet the 155% peak which the storm hydrograph indicates as being a maximum under the assumed conditions.

At the proposed wet well level for dry weather conditions, the 5 pumps will produce a capacity of 268 mgd, or 180% of the presently designed capacity of the plant. The sewage treatment plant has a design capacity of 150 mgd to meet 1970 conditions, but channels and tanks are designed to pass a total flow of 300 mgd.

Space has been provided in the pumping station for a sixth unit which will be installed when necessary to meet the demands of the year 2000.

Operating Economy of Submerged Control

It is obvious that a great saving in power is possible with submerged control at proper design. With a weekly drawdown including storm flow pumpage, with power estimated to cost 9.5 mills per kilowatt hour as an average, the net savings in power cost with submerged control is estimated at $135,000 per year. The total estimated cost of power under present design conditions is $280,000 annually. Therefore, the savings in power cost alone, by virtue of submerged control, is approximately 33% of the total power bill as it would have been without submerged control.

Advantages and Disadvantages

Submerged control permits the following advantages:

(1) Reduced pumping costs.
(2) Complete filling of a major portion of the intercepting sewer system, thus reducing the possibility of hydrogen sulfide formation and attack on the exposed surfaces of the concrete intercepting sewers.
(3) Control of sewage flows in branch lines.
(4) Greater convenience in removing heavy floating material in the wet well as a result of the high flow line.
(5) Savings in the cost of the pumping station construction due to reduced depth of the wet well. Free flow would have required greater capacity below the crown level of the entering intercepting sewers.
(6) Better control of storm water interception at certain points.

The only disadvantage of such a system is the possibility of sludge and grit deposits in the flooded sections of the intercepting sewer system during low flows.

ACKNOWLEDGEMENTS

The 1949 study of submerged control was prepared by Metcalf & Eddy, Consulting Engineers, at the author's direction. The final design of the intercepting sewer system operating under submerged control, except the Turtle Creek section, was prepared by the staff of the Allegheny County Sanitary Authority under the direction of the author. The Turtle Creek section, in addition to others, was designed by Metcalf & Eddy. The flow hydrographs and the hydraulic gradient reproduced in this paper were prepared by the Sanitary Authority staff.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Board of Direction are identified by the symbols (BD). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper numbers are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1449 is identified as 1449 (HY 6) which indicates that the paper is contained in the sixth issue of the Journal of the Hydraulics Division during 1957.

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JULY: 1289(SM3), 1290(EM3), 1291(EM3), 1292(EM3), 1293(EM3), 1294(HW3), 1295(HW3), 1296(HW3), 1297 (HW3), 1298(HW3), 1299(SM3), 1300(SM3), 1301(SM3), 1302(ST4), 1303(ST4), 1304(ST4), 1305(SU1), 1306 (SU1), 1307(SU1), 1308(ST4), 1309(SM3), 1310(SU1)[c], 1311(EM3)[c], 1312(ST4), 1313(ST4), 1314(ST4), 1315 (ST4), 1316(ST4), 1317(ST4), 1318(ST4), 1319(SM3)[c], 1320(ST4), 1321(ST4), 1322(EM3), 1323(AT1), 1324 (AT1), 1325(AT1), 1326(AT1), 1327(AT1), 1328(AT1)[c], 1329(ST4)[c].

AUGUST: 1330(HY4), 1331(HY4), 1332(HY4), 1333(SA4), 1334(SA4), 1335(SA4), 1336(SA4), 1337(SA4), 1338 (SA4), 1339(CO1), 1340(CO1), 1341(CO1), 1342(CO1), 1343(CO1), 1344(PO4), 1345(HY4), 1346(PO4)[c], 1347 (BD1), 1348(HY4)[c], 1349(SA4)[c] 1350(PO4), 1351(PO4).

SEPTEMBER: 1352(IR2), 1353(ST5), 1354(ST5), 1355(ST5), 1356(ST5), 1357(ST5), 1358(ST5), 1359(IR2), 1360 (IR2), 1361(ST5), 1362(IR2), 1363(IR2), 1364(IR2), 1365(WW3), 1366(WW3), 1367(WW3), 1368(WW3), 1369 (WW3), 1370(WW3), 1371(HW4), 1372(HW4), 1373(HW4), 1374(HW4), 1375(PL3), 1376(PL3), 1377(IR2)[c],1378 (HW4)[c], 1379(IR2), 1380(HW4), 1381(WW3)[c], 1382(ST5)[c], 1383(PL3)[c], 1384(IR2), 1385(HW4), 1386(HW4).

OCTOBER: 1387(CP2), 1388(CP2), 1389(EM4), 1390(EM4),1391(HY5), 1392(HY5), 1393(HY5), 1394(HY5), 1395 (HY5), 1396(PO5), 1397(PO5), 1398(PO5), 1399(EM4), 1400(SA5), 1401(HY5), 1402(HY5), 1403(HY5), 1404 (HY5), 1405(HY5), 1406(HY5), 1407(SA5), 1408(SA5), 1409(SA5), 1410(SA5), 1411(SA5), 1412(EM4), 1413 (EM4), 1414(PO5), 1415(EM4)[c], 1416(PO5)[c], 1417(HY5)[c], 1418(EM4), 1419(PO5), 1420(PO5), 1421(PO5), 1422(SA5)[c], 1423(SA5), 1424(EM4), 1425(CP2).

NOVEMBER: 1426(SM4), 1427(SM4), 1428(SM4), 1429(SM4), 1430(SM4)[c], 1431(ST6), 1432(ST6). 1433(ST6), 1434(ST6), 1435(ST6), 1436(ST6), 1437(ST6), 1438(SM4), 1439(SM4), 1440(ST6), 1441(ST6), 1442(ST6)[c], 1443(SU2), 1444(SU2), 1445(SU2), 1446(SU2), 1447(SU2), 1448(SU2)[c].

DECEMBER: 1449(HY6), 1450(HY6), 1451(HY6), 1452(HY6), 1453(HY6), 1454(HY6), 1455(HY6), 1456(HY6)[c], 1457(PO6), 1458(PO6), 1459(PO6), 1460(PO6)[c], 1461(SA6), 1462(SA6), 1463(SA6), 1464(SA6), 1465(SA6), 1466(SA6)[c], 1467(AT2), 1468(AT2), 1469(AT2), 1470(AT2), 1471(AT2), 1472(AT2), 1473(AT2), 1474(AT2), 1475(AT2), 1476(AT2), 1477(AT2), 1478(AT2), 1479(AT2), 1480(AT2), 1481(AT2), 1482(AT2), 1483(AT2), 1484(AT2), 1485(AT2)[c], 1486(BD2), 1487(BD2), 1488(PO6), 1489(PO6), 1490(BD2), 1491(BD2), 1492(HY6), 1493(BD2).

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JANUARY: 1494(EM1), 1495(EM1), 1496(EM1), 1497(IR1), 1498(IR1), 1499(IR1), 1500(IR1), 1501(IR1), 1502 (IR1), 1503(IR1), 1504(IR1), 1505(IR1), 1506(IR1), 1507(IR1), 1508(ST1), 1509(ST1), 1510(ST1), 1511(ST1), 1512(ST1), 1513(WW1), 1514(WW1), 1515(WW1), 1516(WW1), 1517(WW1), 1518(WW1), 1519(ST1), 1520 (EM1)[c], 1521(IR1)[c], 1522(ST1)[c], 1523(WW1)[c], 1524(HW1), 1525(HW1), 1526(HW1)[c], 1527(HW1).

FEBRUARY: 1528(HY1), 1529(PO1), 1530(HY1), 1531(HY1), 1532(HY1), 1533(SA1), 1534(SA1), 1535(SM1), 1536(SM1), 1537(SM1), 1538(PO1)[c], 1539(SA1), 1540(SA1), 1541(SA1), 1542(SA1), 1543(SA1), 1544(SM1), 1545(SM1), 1546(SM1), 1547(SM1), 1548(SM1), 1549(SM1), 1550(SM1), 1551(SM1), 1552(SM1), 1553(PO1), 1554(PO1), 1555(PO1), 1556(PO1), 1557(SA1)[c], 1558(HY1)[c], 1559(SM1)[c].

MARCH: 1560(ST2), 1561(ST2), 1562(ST2), 1563(ST2), 1564(ST2), 1565(ST2), 1566(ST2), 1567(ST2), 1568 (WW2), 1569(WW2), 1570(WW2), 1571(WW2), 1572(WW2), 1573(WW2), 1574(PL1), 1575(PL1), 1576(ST2)[c], 1577(PL1), 1578(PL1)[c], 1579(WW2)[c].

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c],1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

c. Discussion of several papers, grouped by divisions.

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